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HYDROLOGY OF QUIRKE AND PANEL URANIUM TAILINGS BASINS

(AECB Project No. 5.136.1)

by

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and
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Research report

HYDROLOGY OF QUIRKE AND PANEL URANIUM TAILINGS BASINS

A report prepared by R.M. King, Shawinigan Lavalin Inc. and L. MacPhie, Geocon Inc., under contract to the Atomic Energy Control Board.

ABSTRACT

The research project described by this report provides the AECB with an independent assessment of the "saturated tailings concept" for the decommissioning of tailing areas at the Rio Algom Quirke and Panel uranium mines near Elliot Lake in Northern Ontario. Hydrologic and hydraulic modelling for each facility showed the interrelation between the design of the water-level control facilities and the water levels in each cell for design flood and extreme low-flow conditions, taking into account all water-balance components. The estimate of seepage rates through the tailings mass is identified as a critical issue.

RÉSUMÉ

Le projet de recherche décrit dans le présent rapport fournit à la CCEA une évaluation indépendante du concept de submergence des résidus pour le déclassement des aires de résidus des mines Quirke et Panel de Rio Algom, près d'Elliot Lake, dans le nord de l'Ontario. La modélisation hydrologique et hydraulique de chaque installation a permis de mettre en évidence l'influence de la conception des ouvrages de contrôle sur les niveaux d'eau dans chacun des bassins de décantation. Les simulations ont été effectuées pour la crue de référence et des conditions d'étiage extrême, compte tenu de tous les composants du bilan hydrologique. L'estimation du taux de percolation à travers la masse de résidus est identifiée comme un élément critique.

DISCLAIMER

The Atomic Energy Control Board is not responsible for the accuracy of the statements made or opinions expressed in this publication and neither the Board nor the author assume liability with respect to any damage or loss incurred as a result of the use made of the information contained in this publication.

FOREWORD

This report for the Atomic Energy Control Board provides an independent assessment of the expected hydraulic behaviour of the Quirke and Panel uranium mine tailings basins, following close out.

The studies were described in an Interim Report and covered:

- Development of a hydrologic data base;
- Flood estimates;
- Development of a simulation model;
- Simulation of levels and flows for flood and low flow conditions.

The physical parameters for each basin conform to data provided directly by Rio Algom early in the study.

Subsequent to the issue of the Interim Report, the AECB provided Shawinigan with the corresponding hydrological investigations of the Panel and Quirke mine tailings areas, which were carried out for Rio Algom by Cumming Cockburn Limited in 1989.

This Final Report is a re-issue of the Interim Report, with a front section that compares the results of the two sets of hydraulic and hydrologic studies. To avoid extensive re-numbering of the text, this additional text is referred to as Section 0.

The methodology used and the results of this study were presented at a seminar convened by the AECB at their offices on November 1, 1990.

92-05879/

HYDROLOGY OF QUIRKE AND PANEL TAILINGS BASINS

FINAL REPORT

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0. COMPARISON WITH RIO ALGOM ESTIMATES

The primary objective of the current study by Shawinigan Lavalin Inc. (SLI), is to evaluate the probable hydraulic performance of the Quirke and Panel mine tailings facilities in the context of a close out situation, based on designs proposed by Rio Algom.

This subject has been assessed by Rio Algom consultants, Cumming Cockburn Limited (CCL), and that study is described in the CCL reports:

"Hydrological Investigation of the Quirke Mine Tailings Impoundment Areas"

"Hydrological Investigation of the Panel Mine Tailings Impoundment Areas"

dated June 1989.

The following section compares the results of the two studies, as a preface to the assessment provided by Shawinigan for the Atomic Energy Control Board.

0.1 GENERAL

The scope of both sets of studies covers:

- Safety against overtopping, in the event of a very extreme flood. For this criterion both studies provided estimates of the Probable Maximum Flood, and resulting levels estimated to occur throughout each tailings facility.

- Feasibility of maintaining the tailings deposits in a saturated state, without supply of makeup water, based on extreme drought conditions.

The studies were carried out independently. The CCL study in 1989 was made on behalf of Rio Algom, and provides documentation on their proposed close out schemes. The current Shawinigan study was carried out for the AECB. Only very limited information from the CCL report was provided by the AECB. This information was limited to dimensions of weirs (later revised by Rio Algom) and seepage estimates.

The objective of the AECB was that the Shawinigan report would provide a completely independent estimate of the expected hydraulic conditions at each facility.

This section of the report compares basic inputs (dimensions, runoff estimates, etc.) and conclusions from the two studies. It should be noted that as completely different input data, methodology and reporting criteria were used in each study, some approximations may occur in the following comparisons.

0.2 QUIRKE MINE TAILINGS FACILITY

0.2.1 Physical Parameters

The CCL report was prepared in 1989, while Rio Algom were still developing their final close out scheme. Even as this report was prepared, it is understood that the close out scheme has not been completely finalized, particularly with respect to internal and external weirs.

It is apparent that Rio Algom have changed the location of some of the internal dykes, the planned final tailings surface levels and the dimension of the weirs in the intermediate dykes since the CCL study.

The SLI study was based on dimensions given in the Golder report 901-1802 dated March 1990, and weir arrangements were based on briefings by Rio Algom in April 1990.

The CCL tailings basin configuration postdates an earlier proposal by Golder (for Rio Algom) dated March 1989.

It may be noted that Golder/CCL originally numbered the cells from 1 to 5, and the current corresponding numbering by Rio Algom and SLI is 14 to 18.

Essential differences are:

- Internal dykes between cells 15 or 2, 16 or 3, 17 or 4, and 18 or 5 have been relocated, thus changing local drainage areas, dyke lengths (thus seepage) and evaporation.

- Tailings surfaces throughout each cell are changed (note internal dykes are located at contours, thus relocation changes control levels).

- Weir inverts have now been raised by 1.5 feet (dykes 15, 16, 17) to allow for a permeable sub-base in their construction. This affects maximum levels.

- Internal dyke weir geometry (not yet finalized) has been updated from triangular V-notch to trapezoidal with a 20 foot width. The net effect of the raised invert and geometry change is to reduce discharge capacity, apart from leakage through the weir sub-base.

- Freeboard over normal/maximum water levels has been increased from 3 to 4 feet.

The dimensions and areas used for the Quirke mine tailings facility in the two studies are compared in Table 0.1 overleaf.

TABLE 0.1

QUIRKE MINE TAILINGS FACILITY

DIMENSIONS AND AREAS

		SLI	CCL
TOP OF DYKES - FT.			
14 or	1	1314	1313
15	2	1301	1303
16	3	1287	1293
17	4	1274	1283
18	5	1273	N/A
TAILING LEVEL - FT.			
14 or	1	1308	1308
15	2	1295	1298
16	3	1281	1288
17	4	1268	1278
18	5	1266	1268
WEIR INVERTS - FT.			
14 or	1	1311.5	1310
15	2	1298.5	1300
16	3	1284.5	1290
17	4	1271.5	1280
18	5	1268.5	1271.5
WEIR TYPE (CELLS 14-17)		Trapezoidal	V notch
Bottom Width		20	0
Invert at		Tailings + 3.5 ft. 1.5 ft. permeable base	Tailings + 2 ft. No base
DAM G2 SPILLWAY		W = 60, Z = 2	W = 60, Z = 2
NORMAL MAX WATER DEPTH FT.		2	2

TABLE 0.1 (Cont'd.)

QUIRKE MINE TAILINGS FACILITY

DIMENSIONS AND AREAS

	SLI	CCL
SURFACE AREAS $FT^2 \times 10^6$		
14 or 1	6.66	6.58
15 2	3.30	2.37
16 3	5.45	3.23
17 4	1.23	5.50
18 5	1.63	2.70
	18.27	20.38
SUB-BASIN AREAS $FT^2 \times 10^6$		
14 or 1	11.46	9.92
15 2	4.41	3.45
16 3	10.50	4.53
17 4	1.97	6.90
18 5	5.25	4.63
	33.59	29.43

0.2.2 Flood Studies

Both studies adopted the Probable Maximum Flood (PMF) as the design criterion to check the adequacy of the internal and external flow discharge facilities. Having defined inflow hydrographs, both studies used flow routing simulation methods to determine peak levels and discharges in each facility for the design flood conditions.

The impact of a major flood is a function of:

- Peak
- Volume
- Time Base/Hydrograph Shape

As the effect of internal storage is potentially significant, the relative importance of peak and volume can only be assessed by trial.

In the case of the both the Quirke and Panel facilities, the spring PMF, characterized by a larger volume but lower peak than the summer PMF (due to the snow melt contribution but lower rainfall potential), could potentially be a worst case scenario.

The CCL study was based on a summer PMP, while the SLI study developed PMFs for both cases. The results of the routing studies were nearly identical for the two cases with the summer event proving marginally more critical.

Probable Maximum Precipitation (PMP)

Both studies had to "import" empirical data because of the limited hydrometric data base and the very small drainage areas involved.

The CCL study selected the probable maximum precipitation condition (PMP) as being represented by a 12 hour rainfall of 16.7 inches. This widely used criterion is described in a 1957 paper by J. Bruce, and follows experience from hurricane Hazel in Southern Ontario.

The SLI study based the PMP on a recent extensive (1987) Ontario Hydro study for an adjacent basin. That study showed values of:

Duration Hours	Total Precipitation Ins
12	13.4
18	14.7
24	15.4

The SLI study was based on a PMP estimate for 12, 18 and 24 hours. For the SLI study a synthetic hydrograph shape was developed, based on watershed geometry, and after comparison with all recorded hydrograph shapes for peak events in the region.

Both the estimated runoff peaks and volumes assumed by SLI are lower than those selected by CCL.

Considering the summer event probable maximum flood, the SLI estimated flood peak is significantly less than that estimated by CCL, although the precipitation values are of the same order. (A direct comparison cannot be made as SLI values are 3 hour averages). This illustrates the critical importance of the hydrograph shape.

Following further review it is concluded that the PMP/PMF parameters adopted by SLI are reasonable.

Peak Levels

Despite the significant difference in flood peaks, the results of the routing analyses in the two studies are similar. Both studies yielded peak water levels in each cell about 0.5 to 2 feet higher than the normal/maximum level. The SLI increases were in the order of 0.5 to 1.5 feet, while the CCL increases ranged from 1.6 to 1.9 feet.

These results show that:

- The effect of (flood) storage above the normal/maximum water level in each cell is important, as storage volumes are large in relation to runoff volumes.

- The lower SLI flood volumes produced (as would be expected) smaller level increases.

- The effect of the raised, but enlarged, weir inverts was not a significant factor.

- Both studies showed that relatively modest freeboard allowance will be adequate to ensure the safety of the structures.

The results of the flood analyses are summarized in Table 0.2.

0.2.3 Low Flow Analyses

General

A key requirement of the close out scheme is that the tailings deposits should be flooded at all times. The proposed concept is that under normal conditions a minimum depth of 2 feet of water would be maintained. Under flood conditions flows would spill through the system, with the 2 foot depth (more or less) being maintained. Under low runoff conditions the effective live storage in each cell would be slowly drawn down because of evaporation and seepage losses.

The total live storage represented by a 2 foot depth over the tailings surface is very approximately equivalent to several months of evaporation losses, hence long term water balance studies are required to evaluate conditions under critical long term low flow conditions.

Similarly, with negligible runoff over extended periods, internal seepage through the tailings deposits and under the internal dykes is a major component in the water balance for each cell. As the SLI studies demonstrated, it is possible for the upstream cells to dry out, while surplus water is available in the lower cells.

TABLE 0.2

QUIRKE MINE TAILINGS FACILITY

FLOOD ANALYSES - PMF

	SLI	CCL
METHOD	PMP routed with HEC-1 to obtain PMF. ROUT model used for simulations	PMP routed with OTTHYMO
FLOOD VOLUMES		
PMP inches	13.2 14.8 16.0	16.7
Time base hours	12 24 48	12
PEAK TOTAL INFLOWS CFS		
Summer PMF - Peak	1947 (1)	4410
Cfs/sq mile	839 (1)	1901
Spring PMF	1163 (1)	-
(1) 3 hour average		
PEAK LEVELS (SUMMER PMF) FT		
14 or 1	1312.01	1311.6
15 2	1299.88	1301.8
16 3	1285.60	1291.9
17 4	1272.63	1281.9
18 5	1269.25	1272.6 (?)

TABLE 0.2 (Cont'd.)

QUIRKE MINE TAILINGS FACILITY

FLOOD ANALYSES - PMF

REMAINING FREEBOARD (if total freeboard is 4 feet or as noted in [ ])

14 or	1	1.99	2.4
15	2	1.12	2.2
16	3	1.40	2.1
17	4	1.37	2.1
18	5	3.75 [4.5]	3.4 [4.5]

Note: CCL report remaining freeboards were one foot lower than above, as total freeboard was 3 feet for dykes 1-4.

Methods

Both the SLI and CCL studies used a water balance model, however SLI used the same, rather more complex, custom model for the flood routing studies.

The SLI approach was to develop a long term sequence of monthly flows at the site (11 years) and then to patch in an extreme (100 year return period) 120 day sequence in the lowest flow period. The long term simulation showed that water depth in all the cells would be at least the nominal minimum of 2 feet at all times. However the more extreme 120 day events were tested using the initial levels from the start of the driest period in the long term simulation period. No allowance was made for supplementary level control weirs/outlets in the Quirke basin internal dykes. No assessment is made of the total probability of occurrence of this event (combining precipitation and evaporation probabilities), except that it would be significantly lower than 1:100 years.

The CCL approach was to select four alternative 12 month synthetic precipitation sequences and, by one repetition, provide a 24 month precipitation sequence for the water balance simulation for each of the four scenarios. The CCL scenarios corresponded to average, 1:242 year, 1:308 year and 1:880 year events (combining precipitation and evaporation probabilities).

The key elements in the water balance studies are:

- Precipitation/Local Runoff
- Evaporation
- Seepage (Internal - note external is minimal)

No direct comparison has been made of estimated precipitation/local runoff used in each study. As is shown in Table 0.3, SLI evaporation values are lower than those assumed by CCL. Internal seepage amounts are believed to be approximately the same, although widely different assumptions and parameters are used. In both cases seepage under internal dykes is taken as an estimated "most probable" value.

Results

Although there are many different assumptions involved in the two separate studies, the general conclusions are similar.

The CCL study concluded that natural water supply to maintain the tailings in a flooded state would be adequate (or very marginally deficient) for the first two design cases (i.e., up to an effective 300 year return period event). For more severe conditions makeup water and (potentially) a relocation of internal dykes could be required.

The SLI study showed that natural water would be adequate for the 100 year return period dry conditions, provided that most probable internal dyke seepage rates were assumed.

Supplementary analyses showed if seepage rates increased towards the maximum possible value, that the upstream cells could dry out and that, potentially, internal seepage estimates may be the most significant parameter in defining the feasibility of the proposed system.

TABLE 0.3

QUIRKE MINE TAILINGS FACILITY

CONTINUOUS/LOW FLOW SIMULATION

	SLI	CCL
Method	Water balance and flow routing	Water balance and flow routing
Period	1978-88	2 years
Runoff Input	Estimated runoff for total record (takes into account average evaporation over total basin)	Synthetic/selected precipitation for 12 months 3 scenarios
Evaporation	Regional experience verified by Rio Algom records	Amos Quebec data verified by Rio Algom records Average + 3 scenarios

EVAPORATION IN INCHES

	SLI		CCL	
Month	Total Evaporation	Incremental Evaporation Over Water	Average	Design
4			0.87	0.87
5			5.12	6.30
6	6.22	2.94	6.22	8.15
7	7.25	2.98	6.89	5.93
8	4.80	1.56	5.32	5.32
9	3.30	1.45	3.03	3.69
10	2.13	1.75	1.81	1.81
Totals	23.70*	10.68	29.26**	32.07**

* Gross evaporation all surfaces

** Applies only to water surfaces

Critical Flows	Worst of 10 years consecutive monthly flows, with extreme 100 year 4 month patch	Worst case estimated as precipitation 1:40 years Evaporation 1:22 years Joint probabilities: C1 - 1:242 years C2 - 1:308 years C3 - 1:880 years

TABLE 0.3 (Cont'd.)

QUIRKE MINE TAILINGS FACILITY

CONTINUOUS/LOW FLOW SIMULATION

		SLI	CCL
SEEPAGE			
External Seepage		Low	Low
Tailings Permeability Below Internal Dykes	- Base Case	5×10^{-4} cm/sec	5×10^{-3} cm/sec + gradient 1×10^{-2} (1)
	- Worst Case	5×10^{-3} cm/sec	
Seepage Through Permeable Internal Weir Sub-base		Up to 2 cfs at minimum levels	Not applicable

(1) Estimated to yield similar seepage as SLI 5×10^{-4} cm/sec - see SLI report Section 4.6

CONCLUSIONS

SLI

With average/expected seepage (tailings seepage = 5×10^{-4} cm/s) no make up water is required.

If tailings seepage rate is 50% higher than the average rate, makeup water would be required to maintain Cell 14 in a saturated state.

CCL

Scenarios C1, C2 required no/minor makeup water.

C3 requires makeup plus redesign of internal dykes to improve water balance.

0.3 PANEL MINE TAILINGS FACILITY

0.3.1 Physical Parameters

The Panel mine facility consists of two separate natural lake areas, with no intermediate dykes.

As is shown in Table 0.4, the same dimensions (weir inverts, dan crests, etc.) have been used in both studies. There are differences in estimated areas of the tailings surfaces and sub-basins. In most cases these are unlikely to affect the results, however SLI have used a significantly larger value for the South Tailings surface area. This would increase evaporation losses.

0.3.2 Flood Studies

The same comments made in Section 0.2.2 for the Quirke mine facility apply here. The SLI flood volumes and peaks are lower than those estimated by CCL, and they are based on the recent Ontario Hydro studies.

The lower SLI flood runoff volumes result in lower maximum levels in both the Strike Lake and South Tailings impoundment areas.

Both studies provide level increases in the order of 2-4 feet over the initial impoundment levels. The SLI estimates were in the order of 2.4 feet for both basins, while the CCL estimates were in the 3.4 to 4 foot range.

As 5 and 6 foot freeboards are provided at Strike Lake and the South Tailings areas, respectively, adequate freeboard will remain under PMF conditions.

TABLE 0.4

PANEL MINE TAILINGS FACILITY

DIMENSIONS AND AREAS

	SLI	CCL
TOP OF DYKES - FT		
Strike Lake	3365	3365
South Tailings	3322	3322
TAILINGS LEVEL - FT		
Strike Lake	3358	3358
South Tailings	3314	3314
WEIR DIMENSIONS - FT W: Elev.		
Strike Lake	10:3360	10:3360
South Tailings	10:3316 25:3319	10:3316 25:3320
NORMAL MINIMUM WATER DEPTH - FT	2	2
SURFACE AREAS ($FT^2 \times 10^6$)		
Strike Lake	7.31	8.73
South Tailings	6.42	3.93
SUB-BASIN AREAS ($FT^2 \times 10^6$)		
Frayn Lake	10.63	13.50
Strike Lake	16.67	18.12
South Tailings	11.04	12.18

TABLE 0.5

PANEL MINE TAILINGS FACILITY

FLOOD ANALYSES - PMF

	SLI	CCL
METHOD	PMP routed with HEC-1 to obtain PMF. ROUT model used for simulation	PMP routed with OTTHYMO
FLOOD VOLUMES		
PMP inches	13.2 14.8 16.0	16.7
Time base - hours	12 24 48	12
PEAK TOTAL INFLOWS CFS		
Summer PMF - Peak	906 (1)	2785
Spring	541 (1)	1860
(1) 3 hour average		
PEAK LEVELS (SUMMER PMF)		
Strike Lake	3362.31	3363.4
South Tailings	3318.43	3319.9
REMAINING FREEBOARD		
Strike Lake	2.69	1.6
South Tailings	3.57	2.1

0.3.3 Low Flow Analyses

The SLI study assumed initial water storage equivalent to a 2 foot water depth over the tailings deposits. As for the Quirke mine facility, month by month water balance/routing studies were carried out for an 11 year sequence, and an extrapolated 100 year return period 120 day sequence was inserted during the driest year.

As losses are primarily for evaporation, losses are much lower than at the Quirke facility. SLI estimated a 0.7 foot initial storage requirement to avoid exposing the tailings surface, as compared with the assumed 2 feet initial depth. The CCL report estimates losses as less than 1 foot, even for their very extreme drought scenario, and recommend that a minimum one foot depth be allowed for.

Since the CCL report was issued, Rio Algom are proposing a Strike Lake maximum tailings level of 3358 feet i.e., 2 feet below the controlling weir invert level.

TABLE 0.6

PANEL MINE TAILINGS FACILITY

CONTINUOUS/LOW FLOW SIMULATION

	SLI	CCL
Method	Water balance and flow routing	Water balance and flow routing
Period	1978-88	2 years
Runoff Input	Estimated runoff for total record (takes into account average evaporation over total basin)	Synthetic/selected precipitation for 12 months 3 scenarios
Evaporation	Regional experience verified by Rio Algom records	Amos Quebec data verified by Rio Algom records Average + 3 scenarios

EVAPORATION IN INCHES

	SLI		CCL	
Month	Total Evaporation	Incremental Evaporation Over Water	Average	Design
4			0.87	0.87
5			5.12	6.30
6	6.22	2.94	6.22	8.15
7	7.25	2.98	6.89	5.93
8	4.80	1.56	5.32	5.32
9	3.30	1.45	3.03	3.69
10	2.13	1.75	1.81	1.81
Totals	23.70*	10.68	29.26**	32.07**

* Gross evaporation all surfaces

** Applied only to water surfaces

Critical Flows	Worst of 10 years consecutive monthly flows, with extreme 100 year 4 month patch	Worst case estimated as precipitation 1:40 years Evaporation 1:22 years Joint probabilities: C1 - 1:242 years C2 - 1:308 years C3 - 1:880 years

TABLE 0.6 (Cont'd.)

PANEL MINE TAILINGS FACILITY

CONTINUOUS/LOW FLOW SIMULATION

	SLI	CCL
SEEPAGE		
External Seepage	Low	Low
Internal Dykes	Not applicable	Not applicable

CONCLUSIONS

SLI

With average/expected seepage no make up water is required.

Storage required at start of drought:

	Depth (ft)
Strike Lake	0.7
South Tailings	0.1

CCL

Scenarios C1, C2, C3 required no makeup water.

Initial storage required in Strike Lake at start of drought:

Scenario	Water Depth (ft)
C1	0.25
C2	0.50
C3	1.00

1. INTRODUCTION

1.1 PROJECT BACKGROUND

The study described in this report covers hydrologic aspects of two uranium tailings basins, for the Quirke and Panel mines, which are operated by Rio Algom in the Elliot Lake area.

As operation of these mines will cease in the next year or so, procedures for decommissioning the tailings areas at each mine have to be developed by Rio Algom and approved by the Atomic Energy Control Board (AECB).

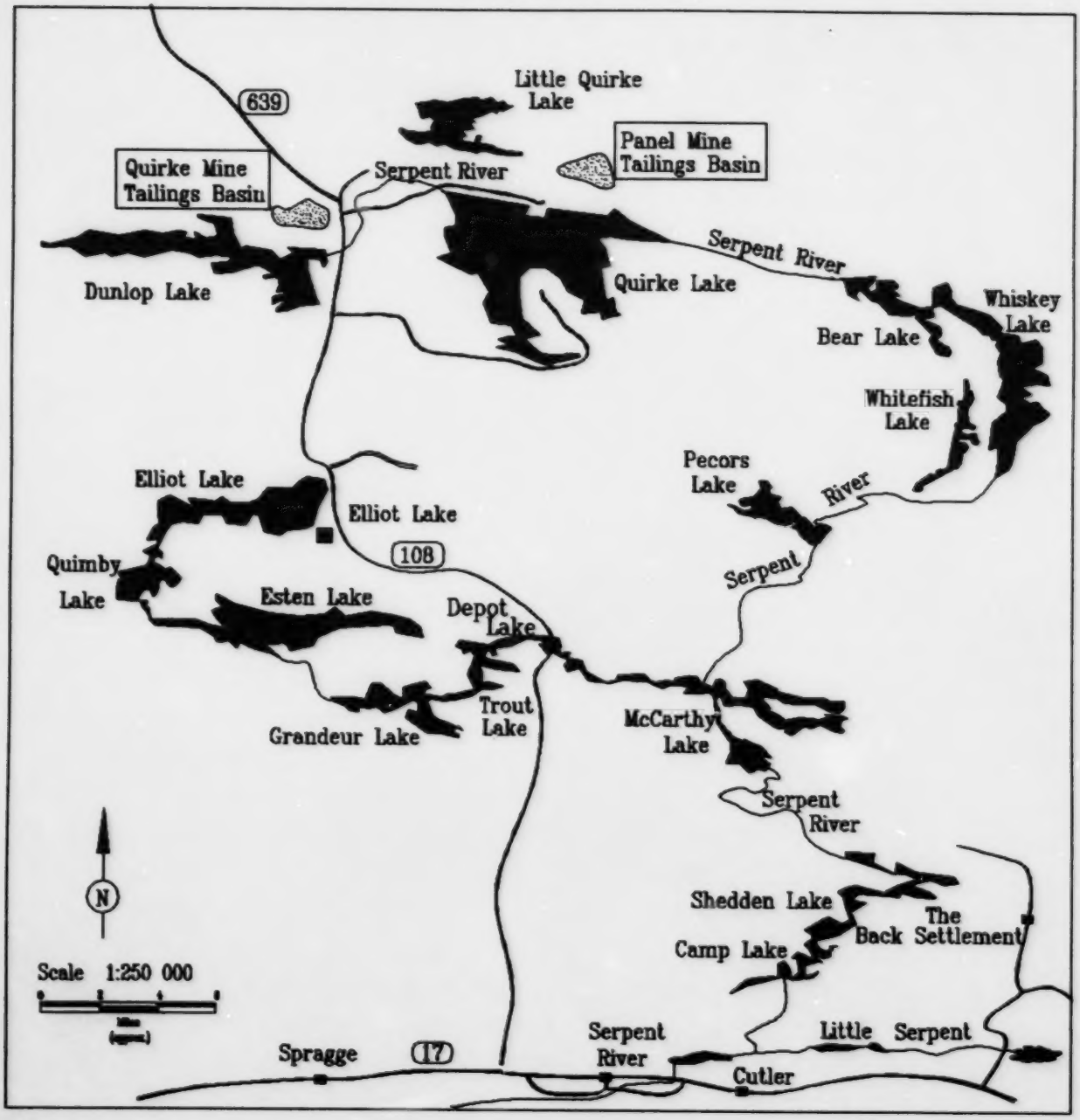
The purpose of this study is to provide the AECB with an independent assessment of the hydrologic aspects of the saturated tailings concept being proposed by Rio Algom for the decommissioning of these sites.

The procedure that is being proposed for both tailings basins is that they be maintained in a flooded state so that the tailings remain saturated, thereby minimizing acid generation, which would otherwise occur in tailings above the phreatic surface (water table). Implicit in this is that natural flows must occur through each basin to maintain the flooding of the tailings deposits.

The close out scheme must be designed for the very long term, with a minimum of maintenance and without requirement for control equipment. Consequently it is necessary for the scheme to be safe against damage from extreme flood events, while at the same time the natural water supply to the basin must be adequate to make up for evaporation and seepage losses during drought conditions.

Figure 1.1

SITE LOCATION PLAN



The hydrologic study described in this report therefore covers, for each basin:

Flood Analyses

- To develop a very extreme flood event (probable maximum flood) for each basin;

- To develop flood flows for a range of frequencies of occurrence.

Low Flow Analyses

- To determine natural runoff for average and severe drought conditions.

Hydraulic Analyses

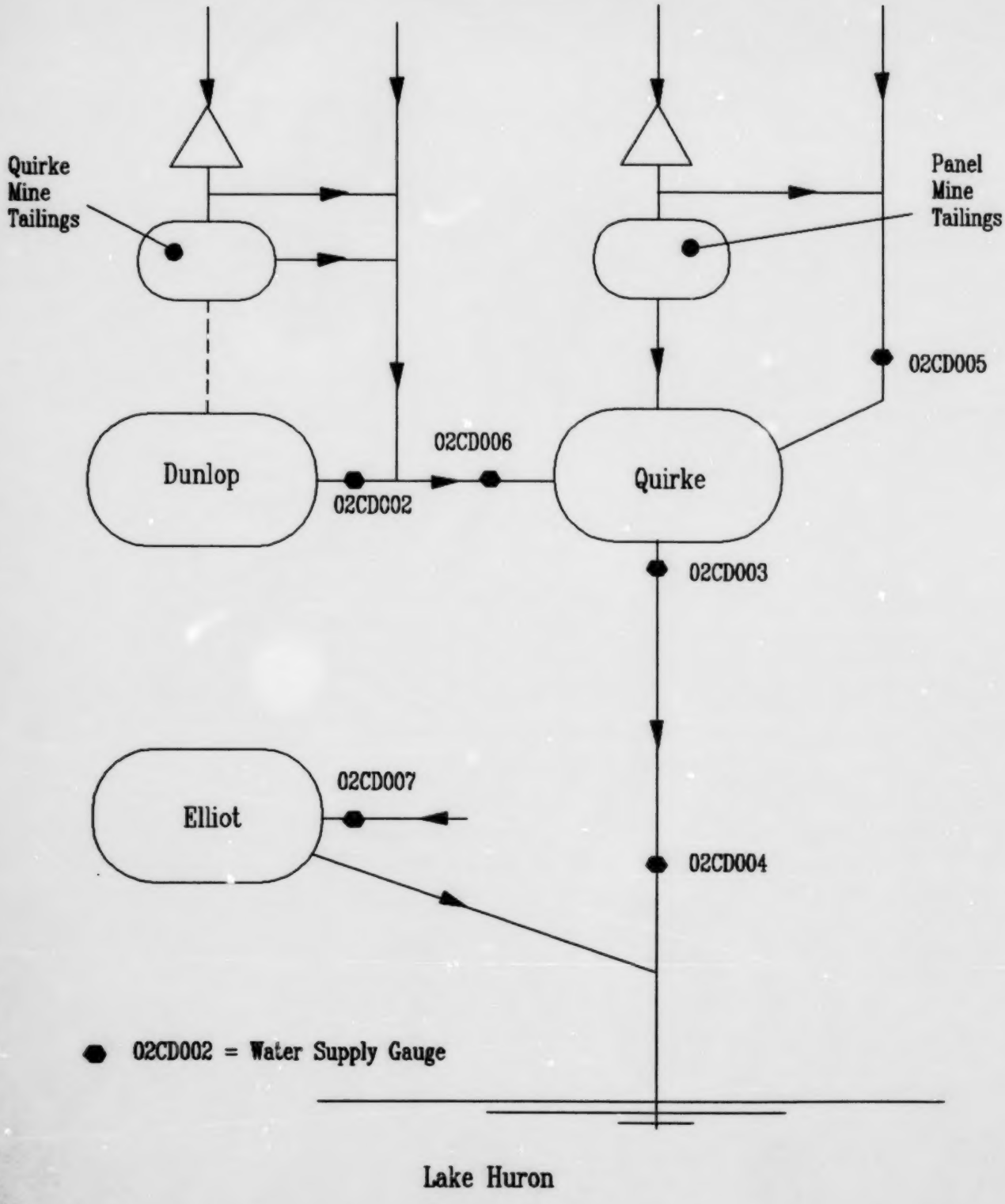
- To route flood flows though the basin, to determine maximum levels and possible overtopping of dykes, in relation to the severity of the flood event;

- To route low flows through each system, to determine minimum levels in each cell, in relation to the severity of the drought event.

The two tailings facilities are described in Section 2, study methodology is provided in Section 3 and 4, and the results of the analyses and modelling are given in Sections 5 and 6 of this report.

Figure 1.2

SERPENT RIVER DRAINAGE BASIN



1.2 TERMS OF REFERENCE

The sections of the terms of reference for the overall study, which describe the objectives, scope of work and tasks to be performed, are quoted overleaf.

AECB RESEARCH PROJECT

HYDROLOGY OF QUIRKE AND PANEL URANIUM TAILINGS BASINS

EXTRACTS FROM WORK STATEMENT

A. Objectives

The objective of this project is to provide AECB staff with an independent assessment on the "saturated tailings concept" for the decommissioning of tailings areas. This information will assist AECB staff in evaluating the adequacy of the licensee's design of the dams' freeboard and spillways capacity.

B. Scope of Work

Based on critical reviews of reports on the Quirke and Panel tailings areas, and of existing models of decommissioning schemes (which in turn are based on hydrogeologic calculations) the contractor will calculate tailings flows and elevations during extreme hydrologic events (i.e., flood or drought) and under normal conditions. In addition, the contractor will compare and contrast his assumptions and calculations with those of the AECB licensee.

C. Tasks to be Performed

1. Review background reports on the Quirke and Panel tailings areas and on their proposed decommissioning schemes.

2. Critically review existing models of hydrologic calculation, with emphasis on their applicability to the two specific tailings areas to be modelled. Select one (or several) models for use in tasks 3 and 4, below. Justify the selection.

3. Estimate input parameters for the model(s) for the two sites. (A field trip to the sites, by the contractor, will be part of this task.)

4. Perform calculations for flows and elevations during extreme events (floods and drought conditions) and also under (average) normal conditions.

5. Prepare and submit a detailed report on tasks 4.1 to 4.4 above.

6. Compare the results of this study and the study previously conducted by the licensee, explain and interpret differences and give an opinion on the respective reliability of the two sets of calculations. (The contractor will be provided at this stage with results of the licensee's calculations.)

7. Write a final report on the whole project, which contains the report of task 5 and the findings from task 6.

8. Present the results of the project at a seminar to be convened at the AECB headquarters.

2. PROJECT DESCRIPTION

2.1 QUIRKE TAILINGS BASIN

The Quirke mine/mill complex is located approximately where Highway 639 crosses Serpent River, some 10 miles north of Elliot Lake (Figure 1.1.).

The Quirke mine tailings basin takes up Manfred Lake, from which the natural drainage is into Serpent River, which flows into Quirke Lake, some 3 miles east of Manfred Lake. The make up water supply for future flooding of the tailings basin is provided from the small creek which drains into Gravel Pit Lake, from which the natural drainage was into Manfred Lake.

The surface area of the tailings deposits in Manfred Lake is approximately 18×10^6 ft^2, and the basin is being divided into five cells, to reduce the volume of the dykes required to maintain the required water levels over the tailings deposits. The arrangement of five cells utilizes most of the existing internal dykes which were constructed during tailings deposition operations for the purpose of "stacking" the tailings, i.e., making the overall tailings surface steeper to increase storage volume.

The internal dykes are located at selected contour levels, from the original upstream beach deposit elevations to the final downstream tailings levels.

The general arrangement of the basin is shown on Figure 2.1. In terms of hydrology and hydraulics, there are two components, Gravel Pit Lake and the tailings basin.

2.1.1 Gravel Pit Lake

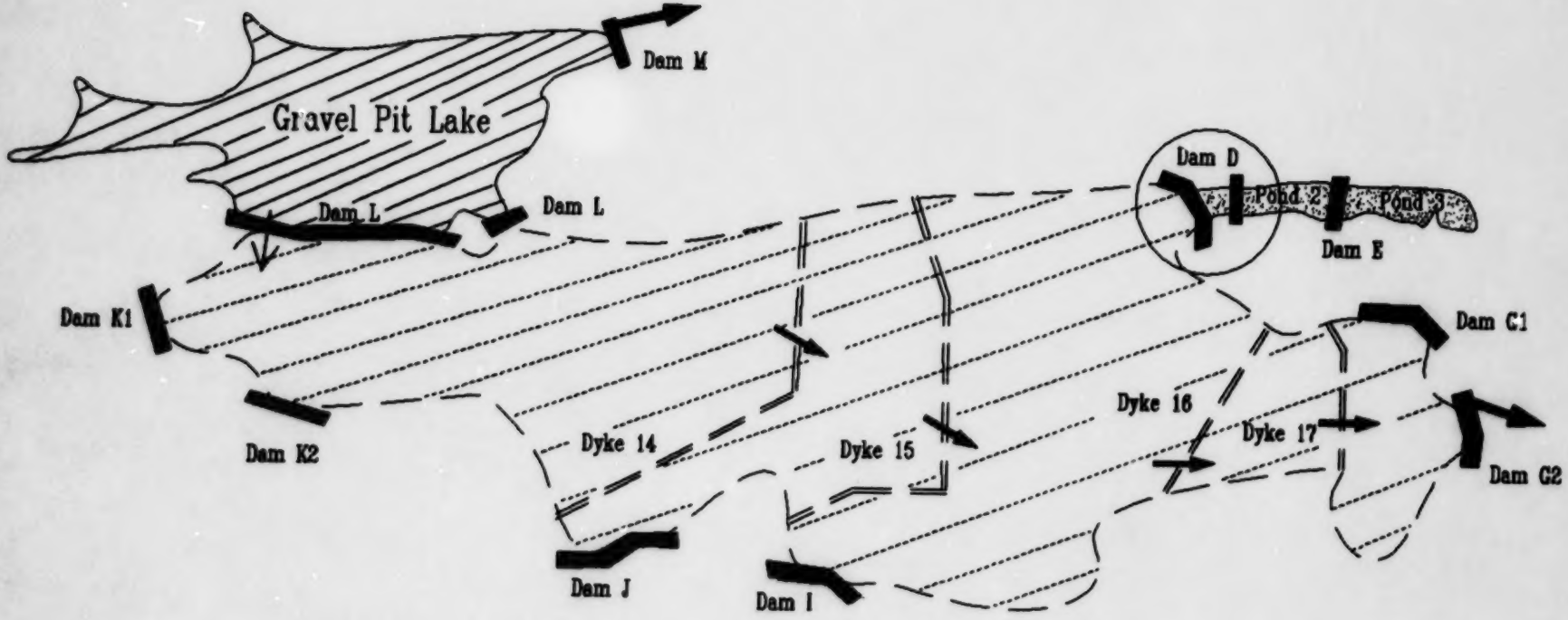
Under natural conditions Gravel Pit Lake runoff discharges into Manfred Lake, which now forms the tailings basin.

Presently the water level in the lake is retained at a normal maximum level of 1315 feet by Dam L, which has recently been raised to elevation 1320, and Dam M which is the spillway/water level control structure, with a weir crest level of 1315 feet.

The primary purpose of the closure structure (Dam L) at Gravel Pit Lake is to impound water on one side and tailings on the other, thus to divert excess natural inflows and flood flows away from the tailings area, which are discharged over Dam M to the Serpent River.

As part of a current program to evaluate possible water supply requirements to the tailings area, to maintain the saturated tailings concept, Dams M and L were raised so that the operating level in Gravel Pit Lake could be raised from 1310 to 1315 feet, to make 485 acre feet of live storage available. Flows are released into the tailings basin as required through two 12 inch pipes with control valves, and with inverts at elevation 1310 feet.

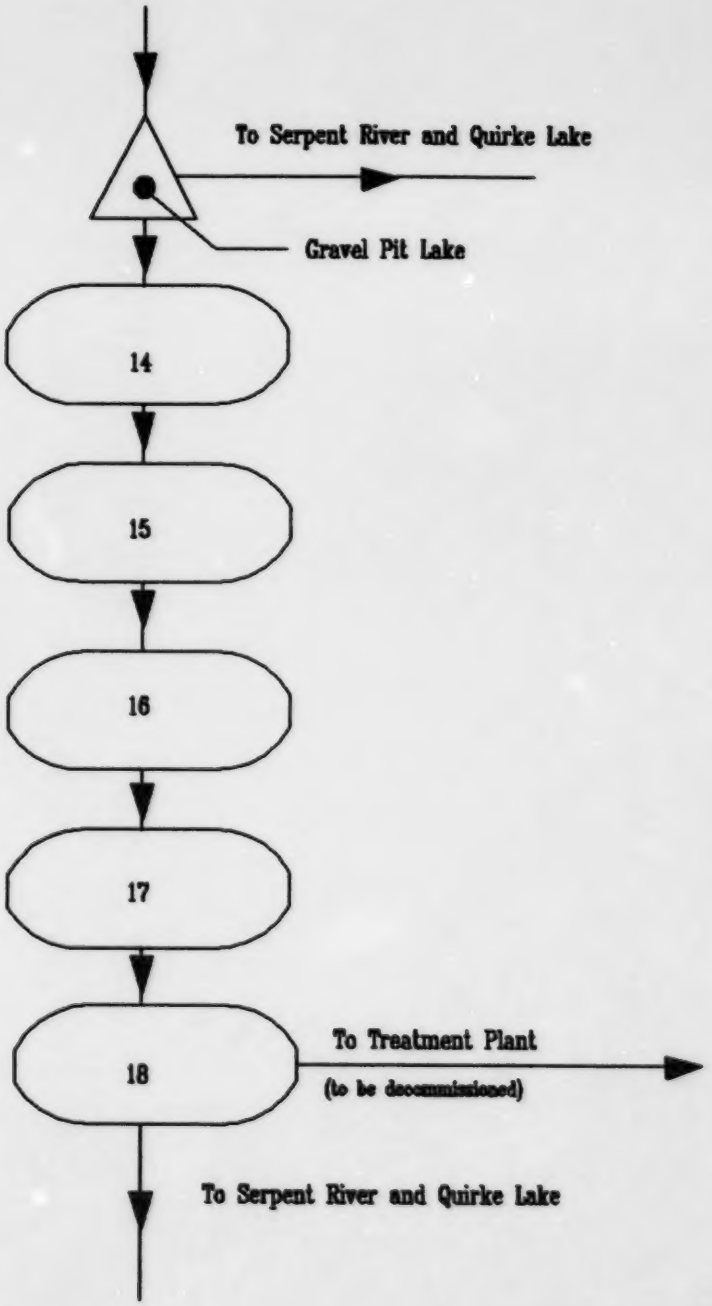
Under future conditions the total discharge in Gravel Pit Lake may be diverted over Dam M. It may be noted that previous studies for the design of Dam M have shown Dam L



Quirke Mine Tailings Basin
General Arrangement

Figure 2.1

DA	SA	NORMAL (MAX) WATER LEVEL	MAX TAILINGS LEVEL
($ft^2 \times 10^6$)		(ft)	(ft)
64.69	4.482	1315	
11.46	6.663	1310	1308
4.412	3.300	1297	1295
10.504	5.449	1283	1281
1.968	1.230	1270	1268
5.254	1.628	1268.5	1266



= Overflow Structure

DA = Drainage Area

SA = Surface Area of Cell

= Cell

Quirke Mine Tailings Basin
Drainage System

Figure 2.2

will not be overtopped under probable maximum flood conditions.

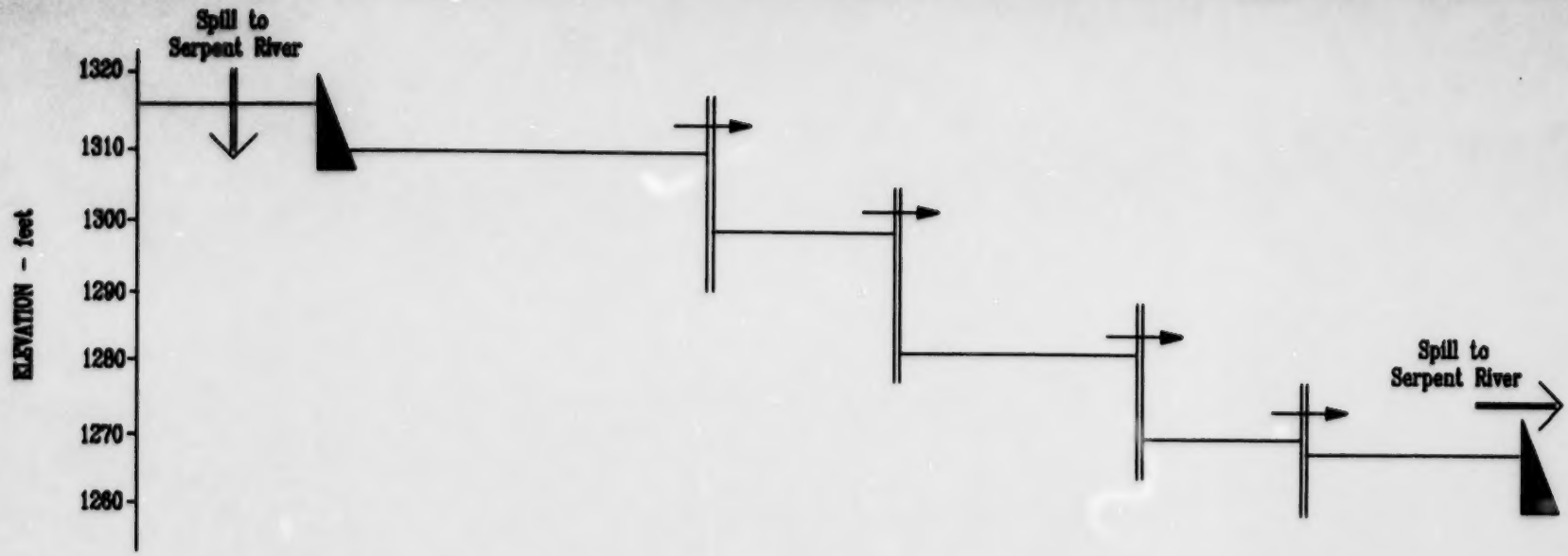
Dam L is an embankment with a thick central core of compacted glacial till founded on grouted bedrock. The core is supported by shells of compacted granular materials. The crest level is 1320 feet and its overall length (both sections) is 1680 feet.

Dam M was designed as an overflow structure and has a low permeability core built of compacted glacial till, founded in part on existing overburden. A service spillway channel with an invert at 1315 feet and width of 15 feet is cut into bedrock on the north abutment of the dam and was designed to pass flows up to the 25 year event. At higher flows, discharge over the dam crest at elevation 1317.5 feet would occur. The length of this crest is 260 feet.

In the design of Dam M it was estimated that under design flood conditions the level in Gravel Pit Lake would rise to 1319 feet, thus leaving one foot of freeboard at Dam L.

2.1.2 Tailings Basin

The closure configuration of the tailings basin, (as presently proposed), comprises a cascade of cells separated by internal dykes with upstream glacial till zones to reduce seepage, and with a series of low permeability external dams (Figures 2.1 and 2.3).



	Gravel Pit Lake	Cell 14	Cell 15	Cell 16	Cell 17	Cell 18
Normal Max. WL	1315	1310	1297	1283	1270	1268.5
Tailings Level	--	1308	1295	1281	1268	1266
Overflow Weir Invert El.	1315	1311.5	1298.5	1284.5	1271.5	1268.5
Top of Dam/Dyke	1320	1314	1301	1287	1274	1273

Quirke Mine Tailings Basin
Levels

Figure 2.3

Internal Dykes

Cell No.	Max. Tailings Surface El. in Cell	Max. (Normal Flow) Water El. in Cell	Containment Dyke/Dam Identification	Final Crest El. of Dyke/Dam
14	1308	1310	Internal Dyke 14	1314
15	1295	1297	Internal Dyke 15	1301
16	1281	1283	Internal Dyke 16 (formerly East Dyke)	1287
17	1268	1270	Internal Dyke 17	1274
18	1266	1268.5	Dams G1 and G2 (External)	1273

The internal dykes already exist (for the most part) but have to be raised, extended and upgraded to the required configuration. The internal dykes are constructed of minus 6 inch rockfill, which is waste rock from the underground operation and which has significant fines. A three foot thick low permeability till seal is provided on the upstream side of the internal dykes to reduce seepage.

Cell 14 (153 acres) is being prepared for flooding. The higher areas of the tailings surface at the west part of Cell 14, and locally along the north and east sides of the cell, are being levelled to elevation 1308. Some areas in the east part of Cell 14 are much lower than elevation 1308 and deeper water will occur in such areas after flooding.

The basic approach to development of Cells 15 to 18 is to flatten the tailings beaches as required to achieve the design elevations and to leave the lower areas as

they are. This will result in flooded cells with a combination of shallow water areas and deeper water areas. Wetlands vegetation will be developed in the shallow water areas.

For "normal flow" conditions a minimum freeboard of 4 feet will be maintained for internal dykes and a corresponding freeboard of 4.5 or 5 feet will be maintained for external dykes. During flood flows the freeboard will be somewhat reduced from the above values.

Flood flows between cells will be handled by shallow overflow spillways. A preliminary sketch provided by Rio Algom showed a 20 foot wide horizontal bottom width and 10 H:1 V slopes. A 12 inch thickness of protective rip-rap is covered with 6 inches of granular B to provide a surface for vehicle access. The need for vehicle access results in the relatively flat 10 H:1 V spillway slopes. The underside of the rip-rap layer will be 4 feet below the crest of the internal dykes, i.e., at the maximum "normal flow" water level in the cell. A smaller flow control outlet to vary water level in the cells below the maximum "normal flow" level and to measure the flows will also be provided (Figure 3.2-A).

It should be noted that the 1.5 foot thick weir sub-base is assumed to be permeable. Therefore under low runoff conditions the additional seepage through this sub-base will tend to maintain levels below the weir invert.

An alternate arrangement by Rio Algom describes a shallow V notch weir for internal dykes (Figure 3.2-B).

External Dykes and Discharges

Surface outflows from the 5 cell system will occur only at Dam G2, which is provided with an overflow weir section in the north abutment. The present overflow weir is a cutoff concrete wall, 60 feet in length and with an invert elevation of 1268.5 feet. This arrangement has been assumed for the close out scheme. It discharges into an excavated chute 60 feet wide with a 2 foot thick rip-rap paving for erosion protection.

Some seepage will occur at each of the external dykes. As a result of the decision to adopt the saturated tailings concept, and the consequent requirement to minimize seepage losses, the Main Dam, which closes a north exit channel from Cell 16, has been reconstructed with a crest level of 1295 feet, thereby allowing water impoundment to elevation 1290 feet in Cell 16.

Seepage at the Main Dam is limited by a thick glacial till core and a bentonite cutoff wall under the dam which extends to bedrock.

Water Balance

Unless additional water is supplied from Gravel Pit Lake, the water balance in each cell will be made up of:

Inflows

- Local runoff, including precipitation onto the cell;
- Seepage through the upstream dyke.

Outflows

- Evaporation;
- Seepage under the downstream internal dyke;
- Seepage through the downstream internal dyke weir sub-base;
- Seepage through external cutoff dykes (if applicable);
- Outflows over downstream dyke weir or dyke crest.

Storage

- Live storage between water surface and top of tailings deposits;
- Storage within the tailings mass.

Under flood conditions the primary elements will be surface inflows and outflows, while under drought conditions the most important elements are storage and evaporation and seepage, as runoff can be negligible for extended periods and seepage is low relative to evaporation.

The conceptual schemes and information on operating levels and local drainage areas are given in Figures 2.2 and 2.3.

2.2 PANEL TAILINGS BASIN

2.2.1 General Arrangement

The Panel Mine is located on the north shore of Quirke Lake, approximately one mile west of Rochester Creek. The tailings basin is centred on Strike Lake, approximately 1.5 miles north of Quirke Lake.

The overall tailings complex includes:

Frayn Lake which is maintained as a fresh water deposit, and which discharges into Rochester Creek. This could provide makeup water to the tailings area if required, by discharging through Dam K.

Strike Lake, which is the main tailings area, and which is, or will be, closed off by perimeter Dams B, E, D and H.

Natural drainage was towards Rochester Creek, and this channel is closed off by Dam B. With the proposed future controlled water level of 3360 feet, drainage would occur into the South Basin at a low saddle on the southwest perimeter of the main tailings area, and this will be closed off by Dam H. However a spillway channel will be excavated between the two basins to discharge excess flood flows out of Strike Lake.

The South Basin, for which the natural drainage to Rochester Creek, is closed off by Dam A. The south basin normal maximum water level is maintained at elevation 3316 feet. Excess flood discharge would spill into an existing channel that leads into Quirke Lake.

The general arrangement of the basin is shown in Figure 2.4

2.2.2 Drainage Areas

The surface and drainage areas of the components of the tailings complex are as follows:

Area	Water Level Ft.	Surface Area ($Ft^2 \times 10^6$)	Drainage Area ($Ft^2 \times 10^6$)
Frayn Lake	3410	-	10.627
Strike Lake	3360	7.309	16.669
South Basin	3316	6.424	11.040

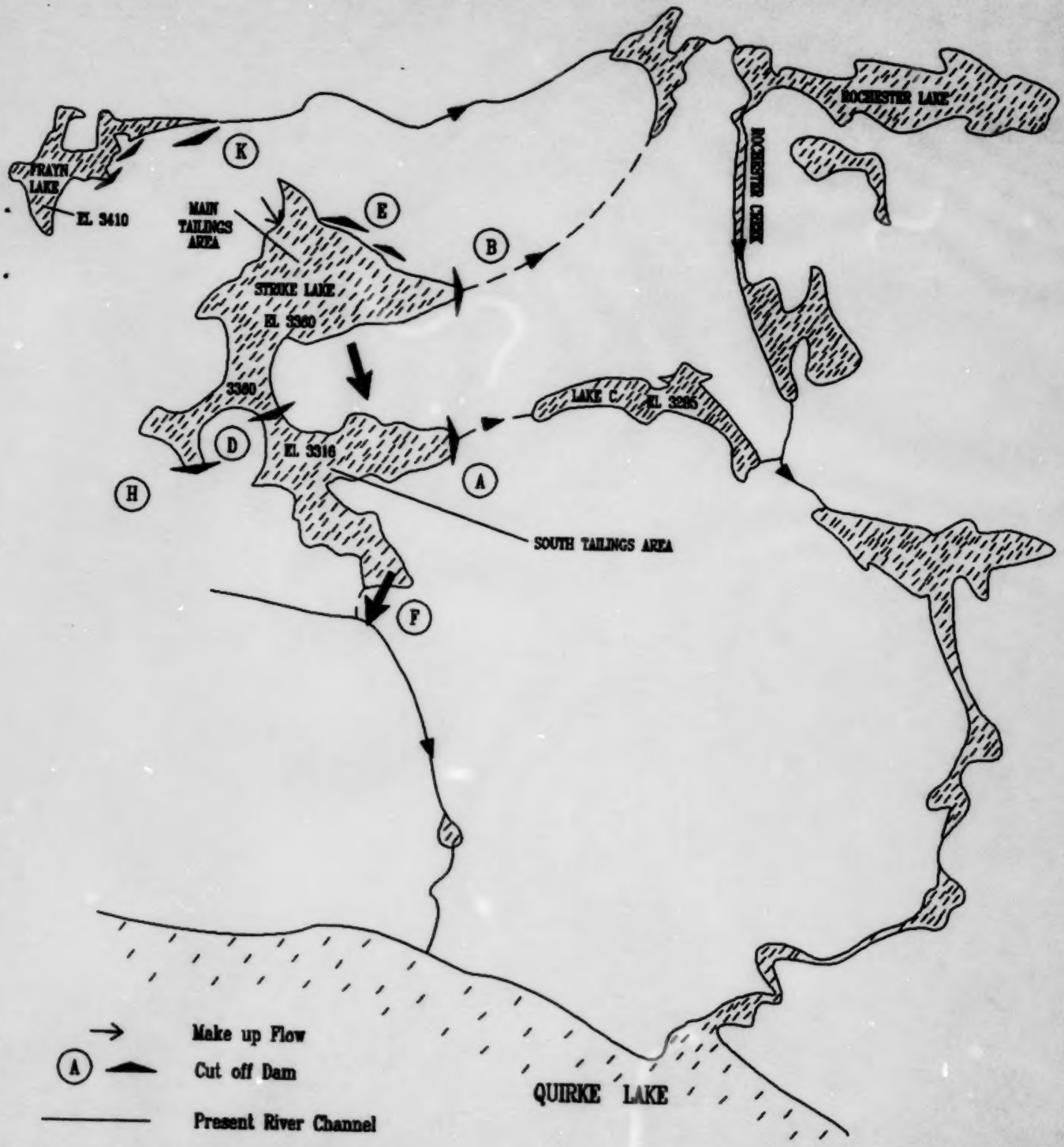
(Levels are given in Panel Datum)

A schematic presentation of the drainage system is shown in Figure 2.5.

2.2.3 Close Out Arrangements

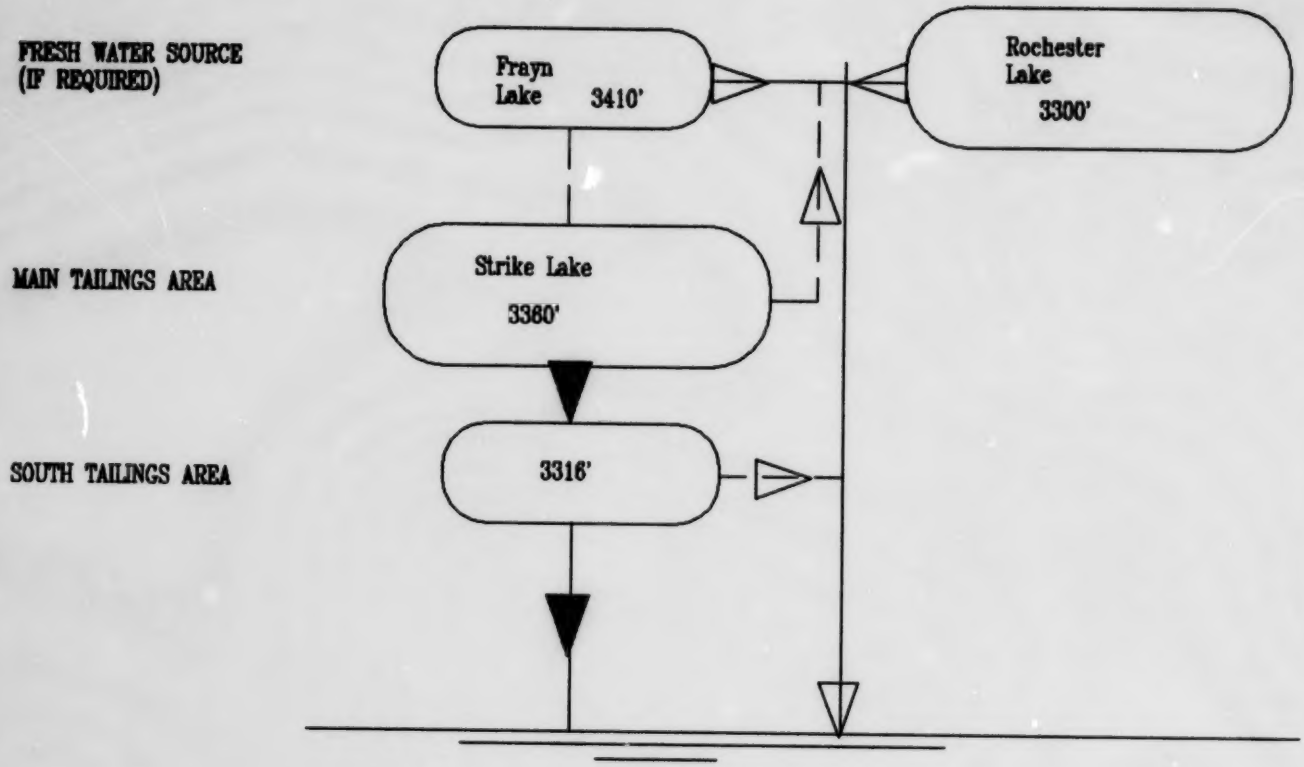
Water Depths

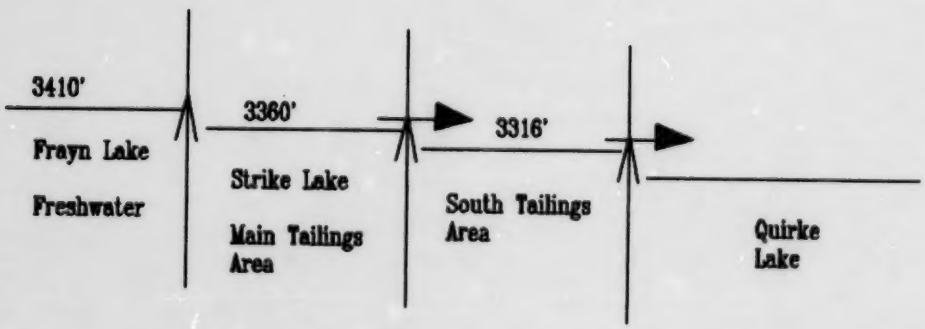
Information on the close out arrangements is limited. It is understood that in the main basin (Strike Lake) where the tailings are deposited, the high tailings beach areas will be levelled to elevation 3358 thus providing a nominal water depth of 2 feet, and the lower tailings surface areas will remain as they are. No internal dykes or cells at different levels are involved. The basin will be flooded to elevation 3360 (or somewhat lower) and wetlands vegetation will be developed in shallow water



Panel Mine Tailings Basin
Schematic Arrangement

Figure 2.4





Former Channel

Present Channel

Spillway

Flow Direction

Panel Mine Tailings Basin
Drainage System

Figure 2.5

areas. The perimeter dams at elevation 3365 will provide 5 feet of freeboard for "normal flow" conditions.

It is presumed that a similar nominal water depth of 2 feet will apply to the South Basin and thus the top of the tailings will be at 3314 feet.

Spillway for Strike Lake

Final dimensions of the proposed spillway channel between Strike Lake and the South Basin have not been provided, except for the invert level of 3360 feet. It will be excavated into rock and thus side slopes will be assumed as vertical. The invert width has been assumed as 10 feet.

Spillway from South Basin

Levels in the South Basin are maintained by a concrete weir, where the overflow discharges into a natural creek leading to Quirke Lake.

Water levels will be controlled by a service weir, 10 feet wide and with an invert elevation of 3316 feet. At high flows discharge occurs over the emergency spillway which has an invert elevation of 3319 feet and a width of 25 feet. The crest elevation of Dams A and F is understood to be at 3322 feet.

2.3 PROJECT DOCUMENTATION

Documentation assembled for the study is listed in Annex A. These comprised:

Reports

Golder
MacLaren
Cumming-Cockburn
Ontario Hydro
Environment Canada

Engineering Drawings

Golder

Flow Records

Water Survey of Canada

Mapping

Energy Mines and Recources - Canada
Ministry of National Resources - Ontario
Rio Algom

3. WATER BALANCE MODEL

3.1 MODEL DEVELOPMENT

3.1.1 General Considerations

The study program includes two primary objectives:

- To determine the system response to peak flood events, and thus to evaluate the risk of overtopping of internal dykes and external dams;

- To demonstrate the risk or probability that tailings deposits will be exposed during extended extreme dry flow periods.

As described below, the approach to obtaining the required information on the behaviour of the Quirke and Panel mine tailing systems for these two conditions has been to use a model that will simulate water balance conditions in each cell/basin component for a sequence of time steps.

In terms of simulation modelling, these two conditions are dissimilar.

The analysis of flood events requires a model that will duplicate unsteady flow conditions with flows and levels varying significantly over short time intervals. Also, while the analyses would be based on water balance methods, parameters such as evaporation and seepage would have minimal significance.

By comparison the simulation of low flow conditions also must be based on water balance, however the evaporation and seepage components will be of major significance in establishing inflow/storage requirements to maintain the tailings in a saturated state.

A major consideration in selecting or developing a model is that the appropriate analytical time steps to simulate each condition would be very different.

For the rainfall component of flood events the time to peak will be less than 1 day, due to the small drainage areas. Due to the rapidly varying flows, and consequent level responses throughout the system, a short calculation time step of 3 hours is appropriate.

However, for the low flow conditions, the availability of storage (initial depth of water over tailings) within the tailings basin suggests that critical conditions may only occur after several months of extreme drought.

A further consideration is that both tailings basins comprise a series of interconnected (water) storage facilities which may include inflows and outflows from several sources. This in turn requires the model to simulate inflow/outflow routing for each cell and for a series of cells.

The selected approach has been to develop a multiple reservoir flow routing model that is appropriate (in terms of data entry and output) for both short term and long term events.

The same model is used for both tailings basins, with that for the Panel mine basin including less components than that for the Quirke mine.

3.1.2 Water Balance

The simulated routing procedure is essentially a water balance process with inflows, outflows and losses being accounted for during each time step on a volume basis.

The various water balance components of a notional two cell basin are illustrated in Figure 3.1 overleaf.

Based on the availability of data and modelling requirements, certain simplifying assumptions are made.

Inflows:

Local surface runoff includes precipitation over the water surface, by adjusting the contributing drainage area accordingly;

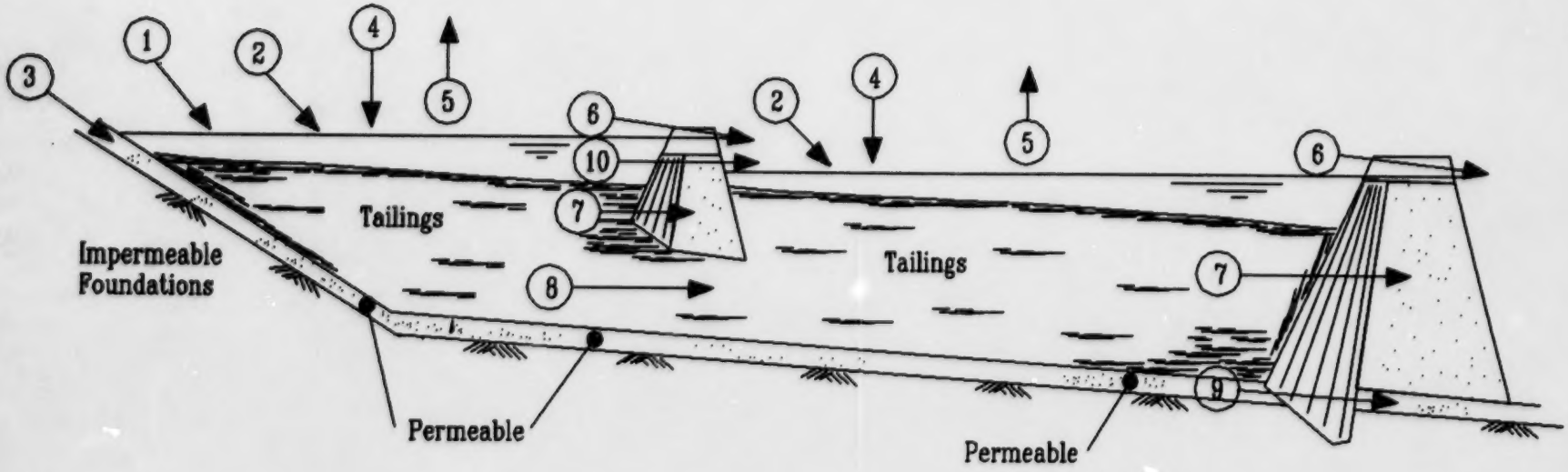
Interflow is assumed to be included in the total estimate of surface runoff (from precipitation).

Outflows:

Evapotranspiration on land is ignored as this is already taken into account in the estimates of surface water runoff yield;

Figure 3.1

Water Balance Components
After Close Out



INFLOW

1. Flow from upstream
2. Local surface runoff
3. Interflow
4. Precipitation

OUTFLOW

5. Evaporation
6. Overflow at weir
7. Core seepage
8. Flow in Tailings mass
9. Foundation seepage
10. Low flow Outlet

Evaporation from the water surface is the difference between total evaporation and average evapotranspiration on land;

An allowance for foundation seepage is included.

Storage:

At the onset of a simulation sequence (dry period or flood) the initial water level is assumed at the normal maximum (usually 2 feet above the highest point of the tailings deposit).

For modelling purposes it is assumed that the tailings surface is horizontal, however to provide the modelling facility of demonstrating water levels lower than the tailings surface, drawdown within the tailings area is assumed. This is calculated at a stipulated porosity (volume of water/total volume).

Make Up Water/Low Flow Outlets:

The model allows for an (optional) constant inflow from upstream into the upstream cell. This is to allow for make up water supply as could be produced from Gravel Lake (Quirke Basin) or Frayn Lake (Panel Basin).

In addition each internal dyke is assumed to have a low flow outlet structure.

(These model features were not used in the study).

3.1.3 Routing

For each time step during the simulation procedure the calculation of water levels in each cell depends directly on the relationship:

Change in storage = inflow - outflow.

Where:

Inflow =
- overflow from upstream internal dyke
- \+ discharge from low flow outlet in upstream dyke
- \+ seepage under upstream internal dyke
- \+ seepage through weir sub-base in upstream internal dyke
- \+ local runoff, including precipitation over the cell surface.

Outflow =
- discharge over downstream internal or external dyke
- \+ discharge from low flow outlet in downstream dyke
- \+ seepage under downstream internal or through external dyke
- \+ seepage through weir sub-base in downstream internal dyke
- \+ evaporation from the cell water surface.

Both inflows and outflows are partially affected by the relative levels between each cell. Inflows and outflows, within the generalized form of the model will include:

Seepage under an internal dyke, which is described as a function of dyke length and differential head.

Spillage through a weir, or over a dyke crest, which is described as a function of upstream level. (Model formulation allows for downstream submergence effect also to be taken into account, however this is a theoretical consideration).

Outflows may also include a nominal or stipulated fixed discharge through a low flow outlet (as presently occurs at Dam L in the Quirke Mine tailings basin). Also seepage losses in the system through an external dam will occur. Based on available information such losses are assumed as fixed for each structure (Section 4.6), and not affected by transient cell water levels.

As cell inflows and outflows are affected by water levels in adjacent cells, an iterative process is used to obtain solutions to the water balance equation for each time step and cell. This applies to the normal situation when the water level is above the tailings surface. When the water level drops below the tailings surface a simplified procedure is used, with discharge being calculated using the initial condition for the time step.

3.1.4 Generalized Model

The model developed for this study provides a time based simulation of the performance and system response of a multiple cascade reservoir string, based on a flow routing, water balance procedure.

Each reservoir can discharge into the next downstream unit by downstream dyke overflow, by low flow outlet, and by seepage. An unlimited number of reservoirs can be included in the string, with the number of reservoirs or cells being specified in the input data. External (to the system) seepage or spillage losses may be specified for any cell.

To allow for efficient simulation of short term and long term events, the routing calculation time step may be varied, e.g., from one hour to one day, and the base inflow hydrograph may be specified for each time step or as average values over a longer period (in this case flows for each time step calculation are interpolated).

As evaporation can vary significantly over the long term (Section 4.5), it may be provided to the same level of detail as the basic flow sequence.

The (model) calculation components related to weir/dyke overflow discharge, seepage and evaporation are covered in the following subsections.

3.1.5 Weirs

The tailings facilities comprise several alternative forms of weirs.

Quirke Basin

Dyke crest overflow	-	rectangular broad crested weir
Internal dyke level control	-	V notch or trapezoidal broad crested weir with permeable sub-base in dyke
Discharge at Dam G2	-	Trapezoidal broad crested weir
Discharge at Dam M	-	Rectangular broad crested weir

Panel Basin

Spillway from Strike Lake	-	Rectangular broad crested weir
Level control at South Basin	-	Rectangular broad crested weir
Emergency spillway at South Basin	-	Rectangular broad crested weir

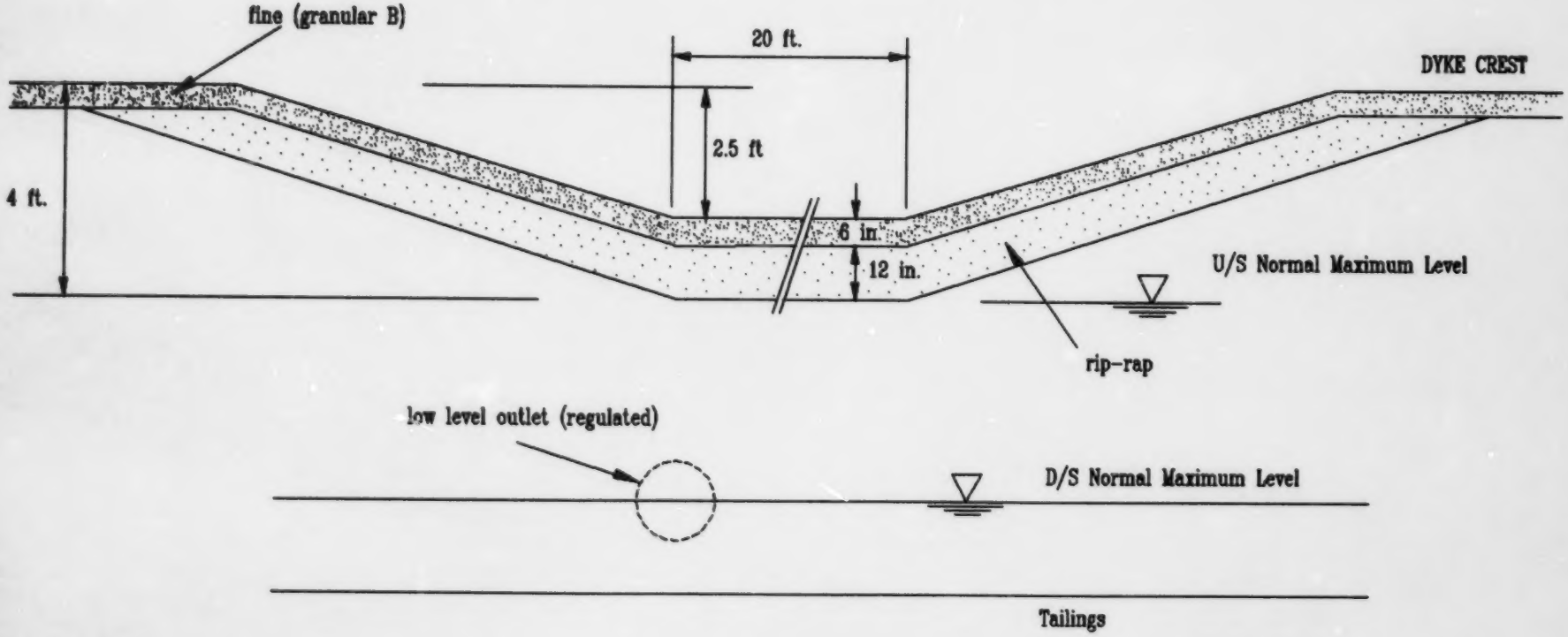
For the purpose of the model only one type of discharge conveyance function is required to cover the above cases.

The standard trapezoidal weir formula adopted for the model can reduce to either a rectangular or triangular section by setting the appropriate parameters to zero.

The standard forms of the weirs are shown in Figure 3.2. As is shown the weir crest is made up of a protective 12 inch layer of rip-rap covered by compacted fine granular material.

Figure 3.2 (a)

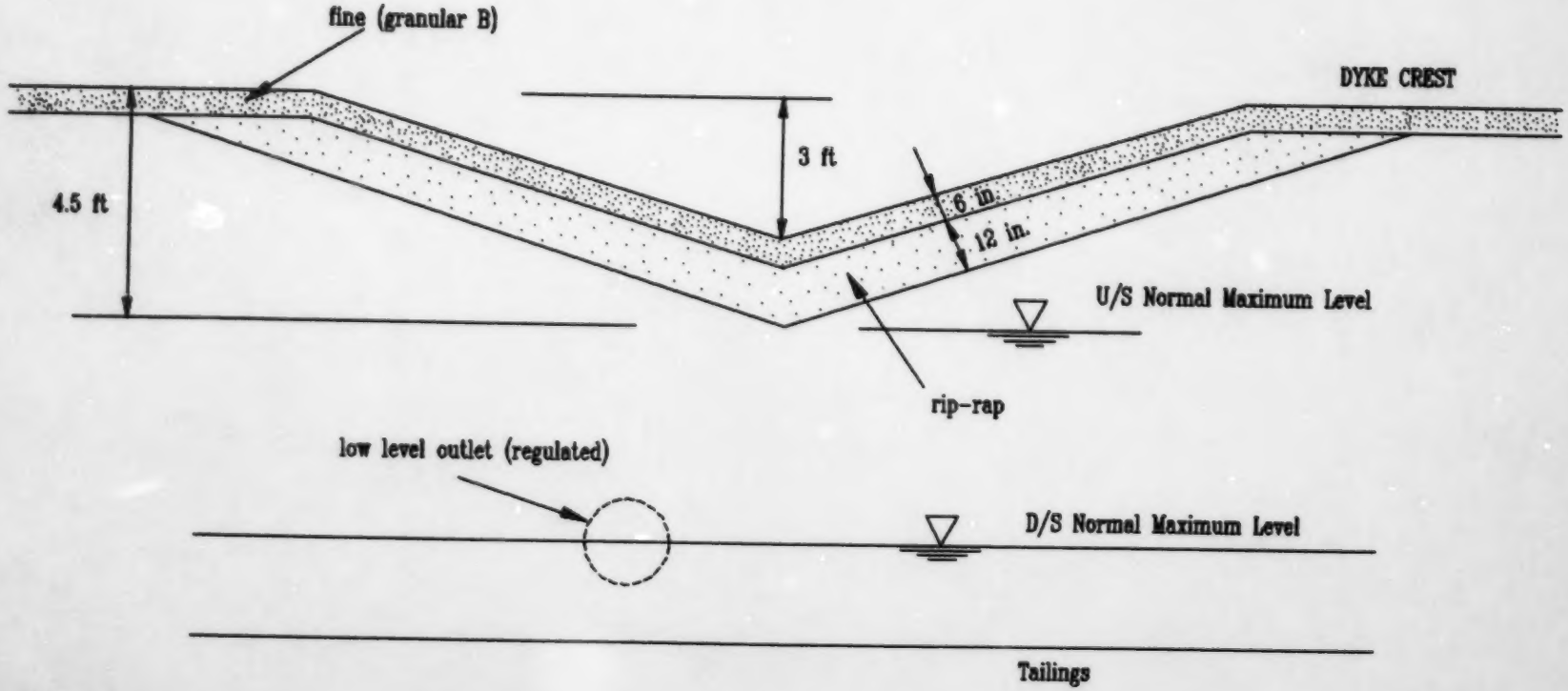
Trapezoidal Weir in Internal Dyke



Schematic Diagram of flow control works between cells

Figure 3.2 (b)

V Notch Weir in Internal Dyke



Schematic Diagram of flow control works between cells

Standard sections are assumed for the weirs in the internal dykes, as these have not yet been constructed or finalized. The weirs at Dam M, G2 (to be lowered later) in the Quirke basin and at the exit of the South Basin at the Panel complex are special cases as they have been constructed. Also the spillway channel at the Panel complex between Strike Lake and the South Basin remains to be sized.

Discharge Estimates

There are two approaches to defining discharge from a broad crested trapezoidal weir. Either a function combining discharge for a rectangular and triangular weir may be used, or discharge may be calculated assuming critical depth.

A. Weir Function

The standard discharge function for a trapezoidal weir with a horizontal crest length L, side slopes, horizontal to vertical, of $h/_v = Z$, and head (H) over the invert is:

$$Q = C_1 LH^{3/2} + C_2 ZH^{5/2}$$

The first term applies to the rectangular section and the section term to the triangular side section.

If L is set to zero, the formula applies to a V notch weir.

Assuming a rounded upstream corner, to prevent constriction, and sufficient slope (upstream to downstream) on the crest to offset friction, flow occurs at critical depth. For the rectangular weir component, the discharge coefficient is 3.087. Consequently C_1 is set to 3.1.

For the triangular weir component the theoretical coefficient is 2.5, and for the broad crested condition this is reduced to 2.45.

The discharges from the trapezoidal or triangular weirs are therefore calculated from the following formula:

$$Q = C_v K_s \left[3.1\ L\ (H-H_b)^{1.5} + 2.45\ Z\ (H-H_b)^{2.5}\right]$$

in which C_v is a small correction for velocity of approach, and which is assumed as 1.0 for the present study, L is the bottom width of the weir, H is the elevation of the water surface just upstream of the structure, H_b is the elevation of the weir crest, Z is the side slope of the weir (horizontal/vertical) and K_s is the submergence correction due to the downstream tailwater elevation (H_t), i.e.,

$$K = 1.0 - 27.8 \left[\frac{H_t - H_b}{H - H_b} - 0.67 \right]^3$$

Submergence is assumed to occur if the downstream excess tailwater depth exceeds 2/3 of the head over the crest, i.e.,

if $(H_t - H_b)/(H - H_b)$ is greater than 0.67

Otherwise K_s is set as 1.0.

B. Critical Flows

The alternative approach to estimate discharges is to assume that critical depth occurs at the weir invert, and to relate the corresponding critical flow to upstream water level. Discharges may be calculated from the following relationships:

$$Q_c = \frac{5.67\ LD_c^{1.5}\ (L + 2D)_c^{0.67}}{(L + 2D_c)^{0.5}}$$

$$D_c = \frac{(4ZH - 3L + 16ZH^2 + 16ZHL + 9B^2)^{0.5}}{10Z}$$

C. Comparison

The derived discharges from the two methods may be compared using a bottom width (L) of 20 feet and a side slope (Z) of 10.

Head Over Crest Ft	Weir Formula Cfs	Critical Flow Cfs
.5	26	37
1.0	86	106
1.5	181	198
2.0	313	310
2.5	487	438

For the purpose of this model the weir formula has been adopted to estimate discharges for all weirs. Assumed discharge coefficients are therefore:

All rectangular sections	3.1
Triangular section	2.45

For the theoretical case of dyke crests being overtopped the discharge coefficient is also assumed as 3.1 and the dyke is treated as a rectangular weir where:

$$Q = 3.1 \, L \, H^{3/2}$$

Typical rating curves are shown in Figure 3.3. Discharges exclude seepage through the weir sub-base, for internal dykes.

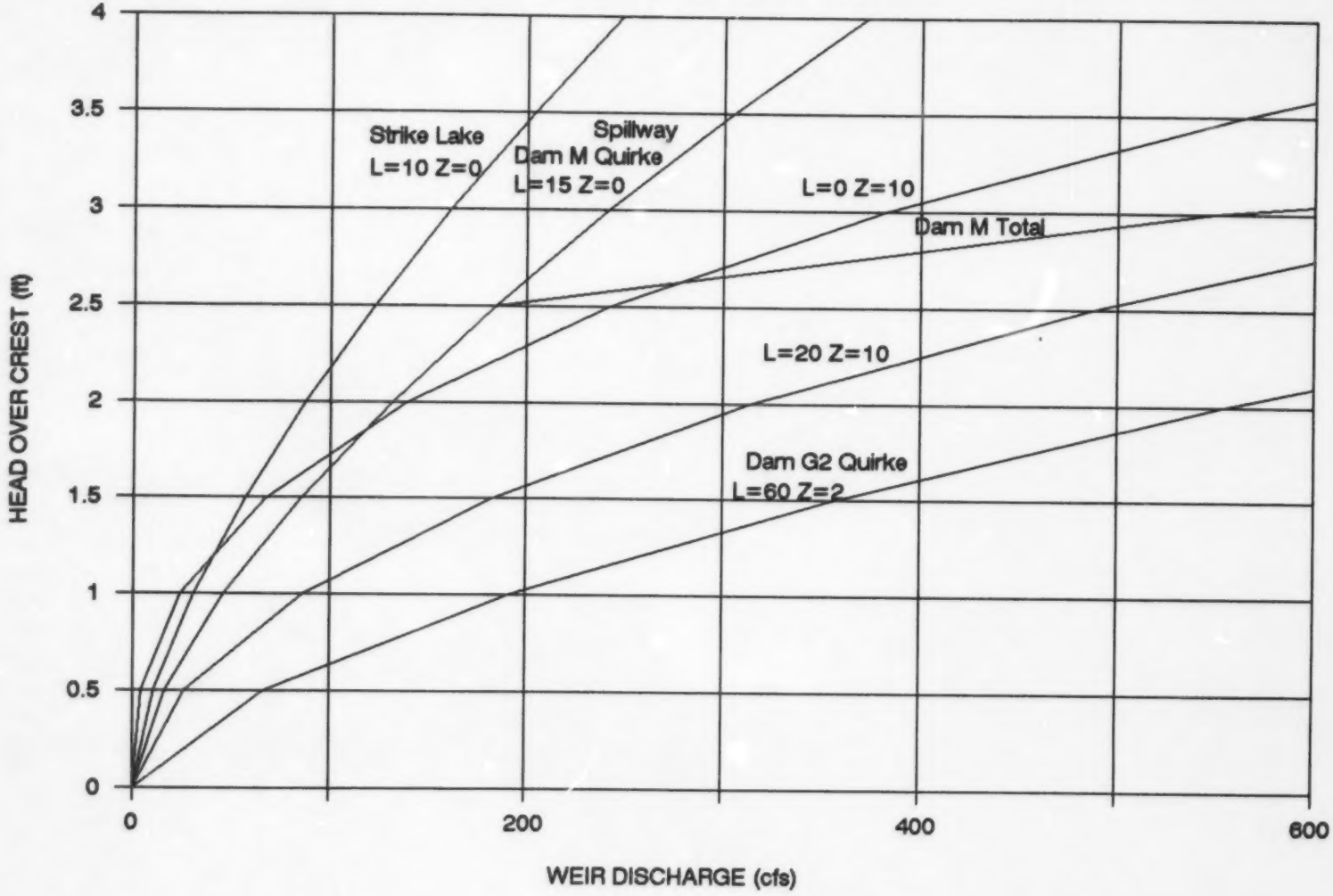
Seepage Through Weir Crest Protection

Trapezoidal weirs are assumed for Dam M and Dam G2 in the Quirke mine tailings basin. Both of these are assumed to have impervious inverts. The Dam M weir is excavated in rock, and the G2 spillway invert is a concrete cutoff wall.

For the trapezoidal weirs in the internal dykes additional flow would occur through the protective invert surface, made up of 6 inches of fine granular material and 12 inches of rip-rap. (Figure 3.2a)

Figure 3.3

WEIR DISCHARGE CURVES



Seepage may be estimated using the formula:

$Q = KgiA$

Where:

Kg = coefficient of permeability

i = hydraulic gradient

A = area of flow

For the rip-rap material the permeability may be in the order of 0.5 to 4 feet per second.

Considering the case of the typical trapezoidal weir, length 20 feet, with 18 inches of rip-rap and granular material and a permeability coefficient of 2 fps, corresponding discharges could be of the following order:

H Ft	Seepage Cfs	(where H is head over underside of the crest protection layer)
1.5	2.1	
2.0	4.5	
2.5	8.8	
3.0	15.6	

These values are insignificant in relation to flood overflows however are significant in routing low flows. This component has been included in the weir discharge characteristics, and has been applied during the low flow simulations.

3.1.6 Seepage

The model takes into account seepage under internal dykes and seepage losses from external dams. The basis for the seepage estimates is given in Section 4.6.

Internal Dykes

For the internal dykes the estimates are based on the relationship:

$Q = FKHL$

Where:

F = shape factor for flow through tailings under low permeability dyke without cutoff

K = permeability of tailings with a range of 1.64×10^{-4} to 1.64×10^{-5} fps (5×10^{-3} to 5×10^{-4} cm/sec)

H = differential head across dyke

L = length of internal dykes

External Dykes

The estimated losses at external dams/dykes is described in Section 4.6. These losses are low in relation to evaporation and are therefore allowed for as an approximation using the above function, and are not reflected in data listed in Tables 3.1 and 3.2.

3.1.7 Evaporation

Evaporation losses, in addition to the evapotranspiration over the whole drainage area, are calculated as a direct function of total evaporation in feet/day applied to the water surface area of the cell or lake. Daily volume losses are then corrected to the time step being used.

Unit evaporation is entered with the runoff input array and values are interpolated for each time step.

The estimates of unit evaporation are described in Section 4.5.

3.1.8 Program Structure

The computer model developed for these analyses consists of a main program and a series of subroutines. The structure of the program and logical relationship between subroutines is shown in Figure 3.4. The program flow chart follows in Figure 3.5.

Individual subroutine functions are summarized as follows:

MAIN	-	Calls routine to read input data
	-	Calls routing main subroutine
	-	Increments reservoirs and time steps
INPUT	-	Reads common data, and data for each cell
RESVROUT	-	Calculates reservoir volume/level changes for each interval

INFLOW	-	Calculates total inflow to each cell
OUTFLOW	-	Calculates total outflow from each cell
FINDIND	-	Searches a look up table/index and interpolates required value
SPILL	-	Calculates spillway discharge
BINARY	-	Solves for reservoir level by iteration using a binary search technique

Figure 3.4
Program Structure

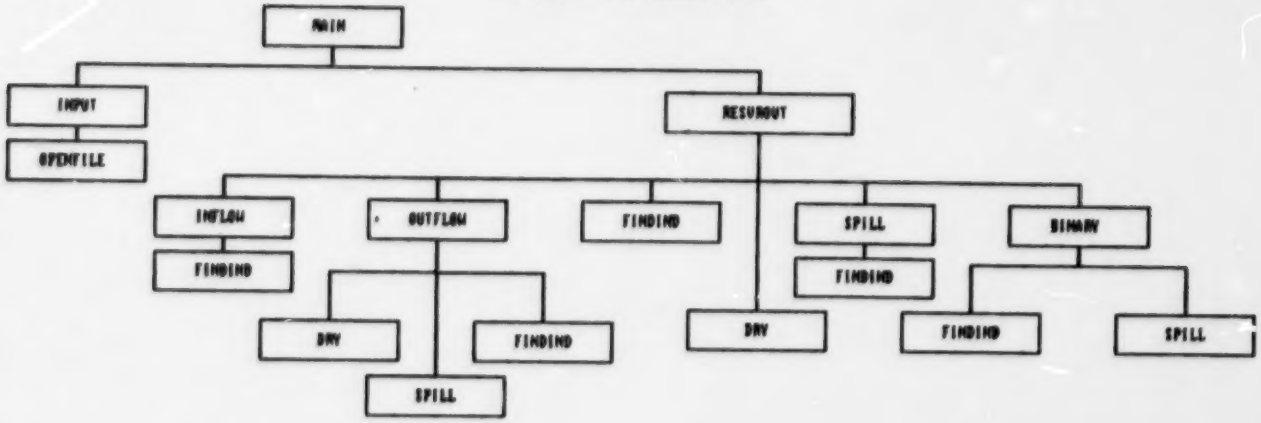


Figure 3.5 (1/7)
Flow Chart - MAIN

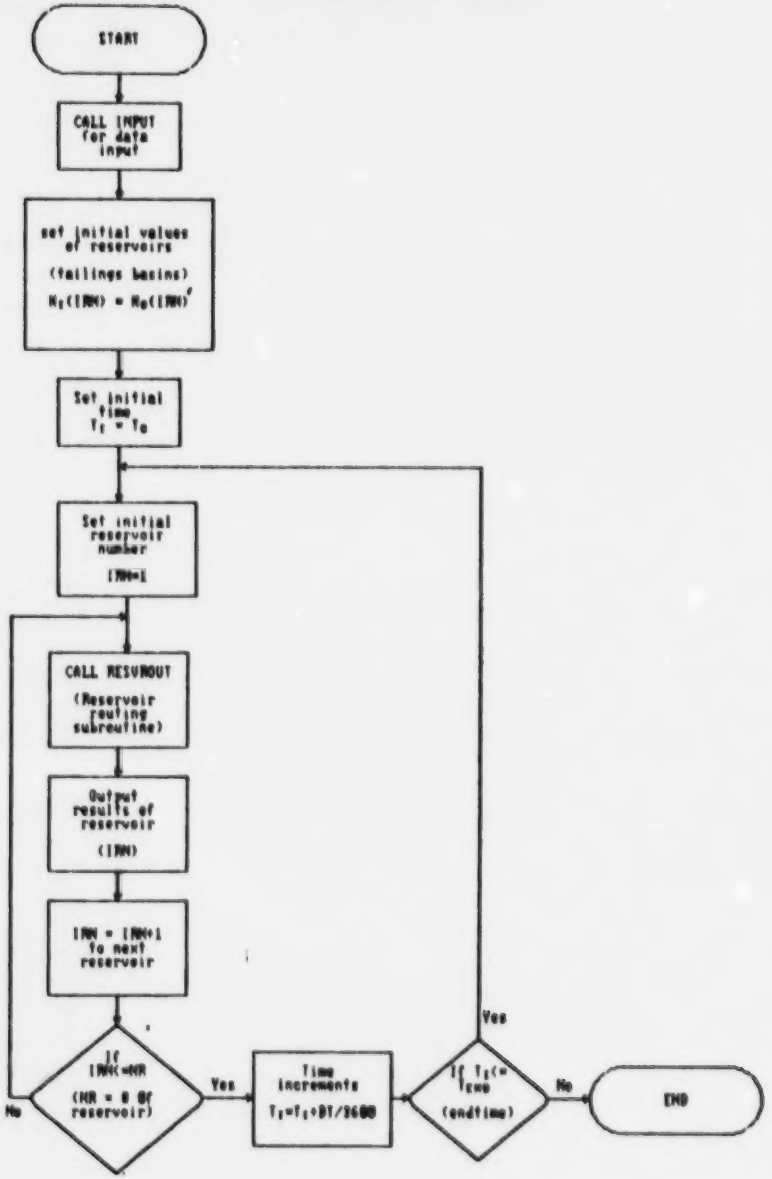


Figure 3.5 (2/7)
Flow Chart - RESVROUT

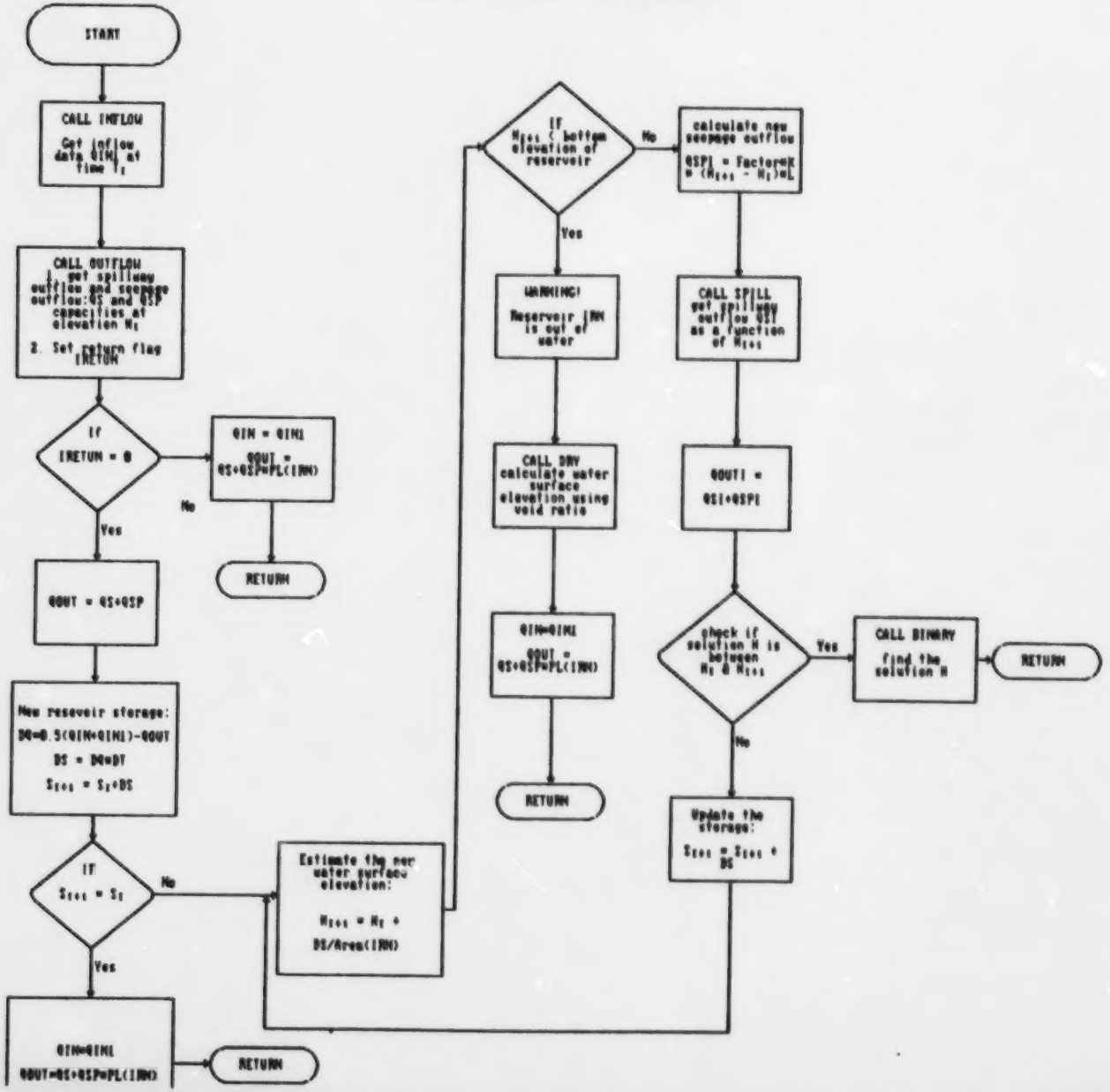


Figure 3.5 (3/7)
Flow Chart - INFLOW

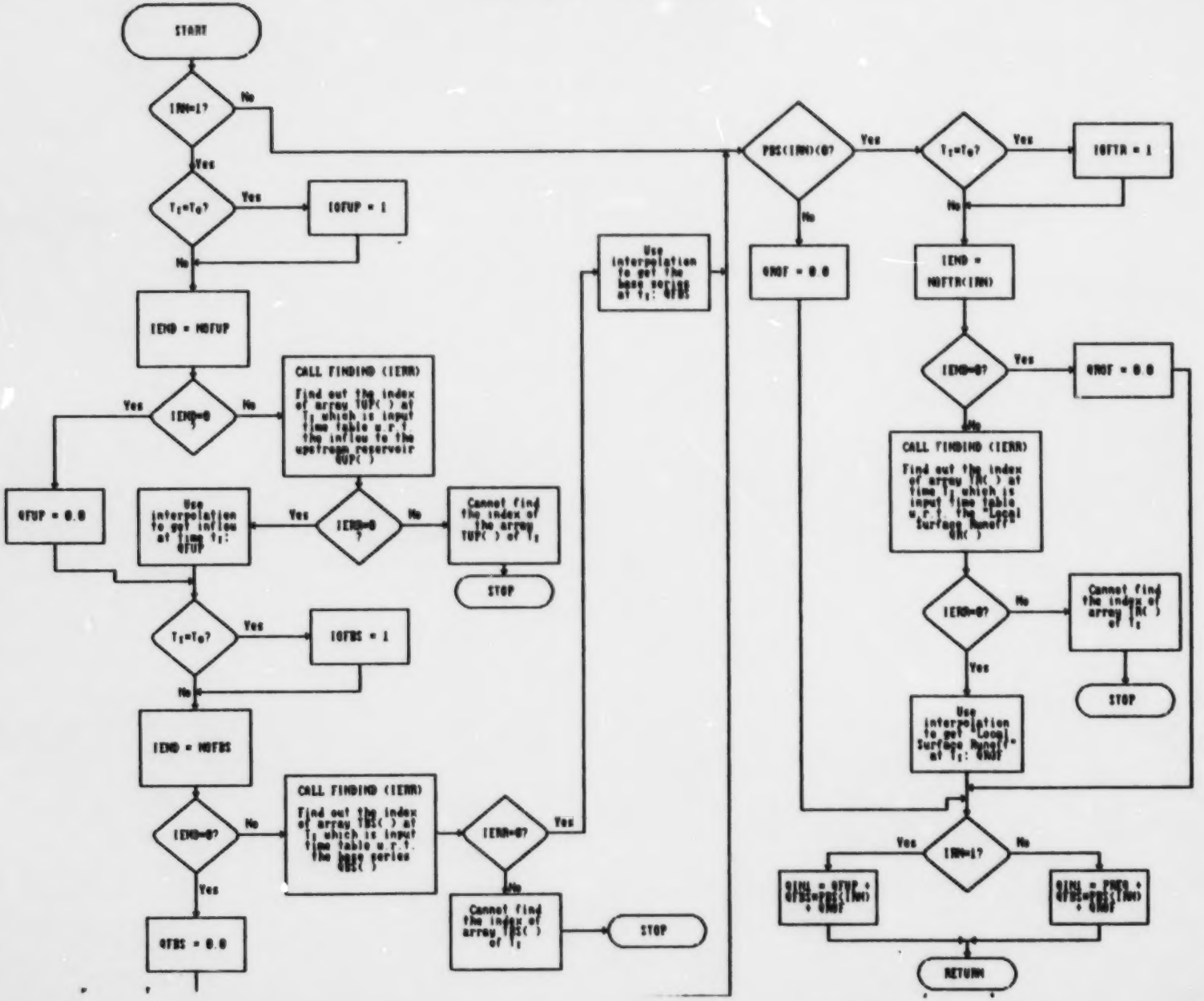


Figure 3.5 (4/7)
Flow Chart - OUTFLOW

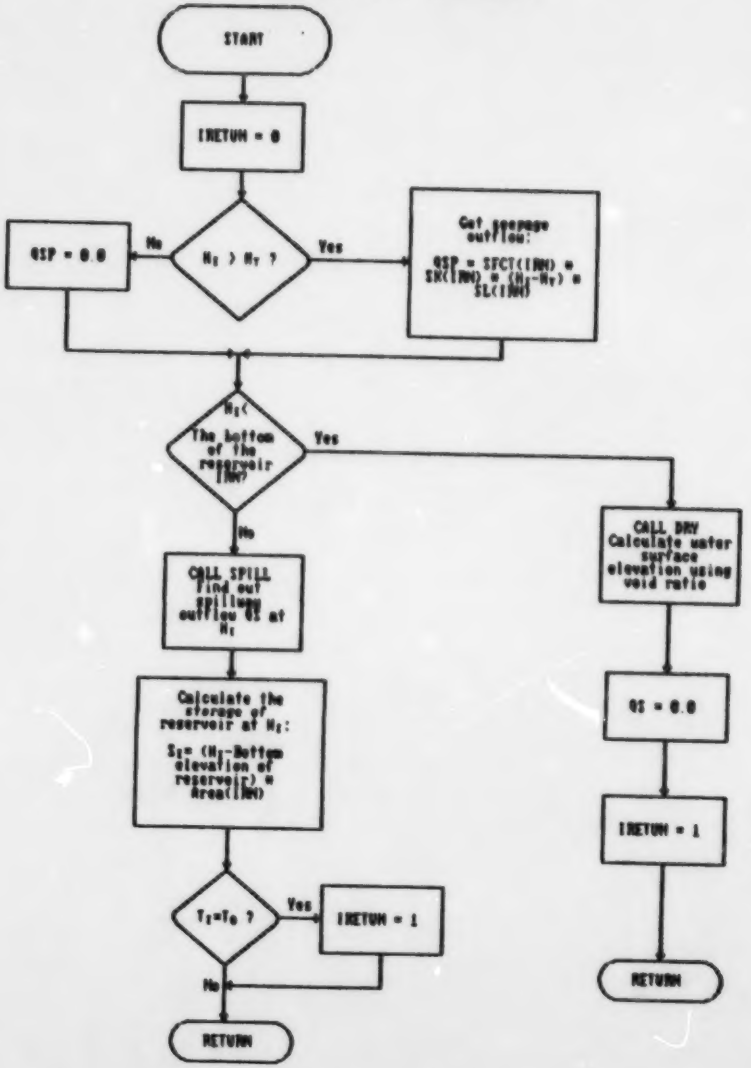


Figure 3.5 (5/7)
Flow Chart - FINDIND

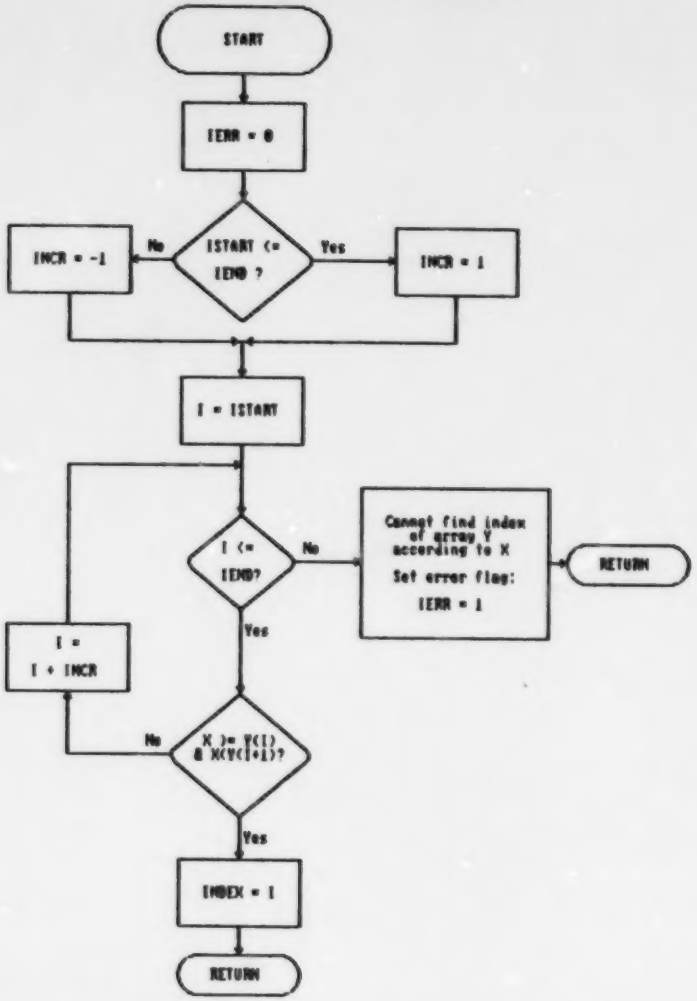


Figure 3.5 (6/7)
Flow Chart - SPILL

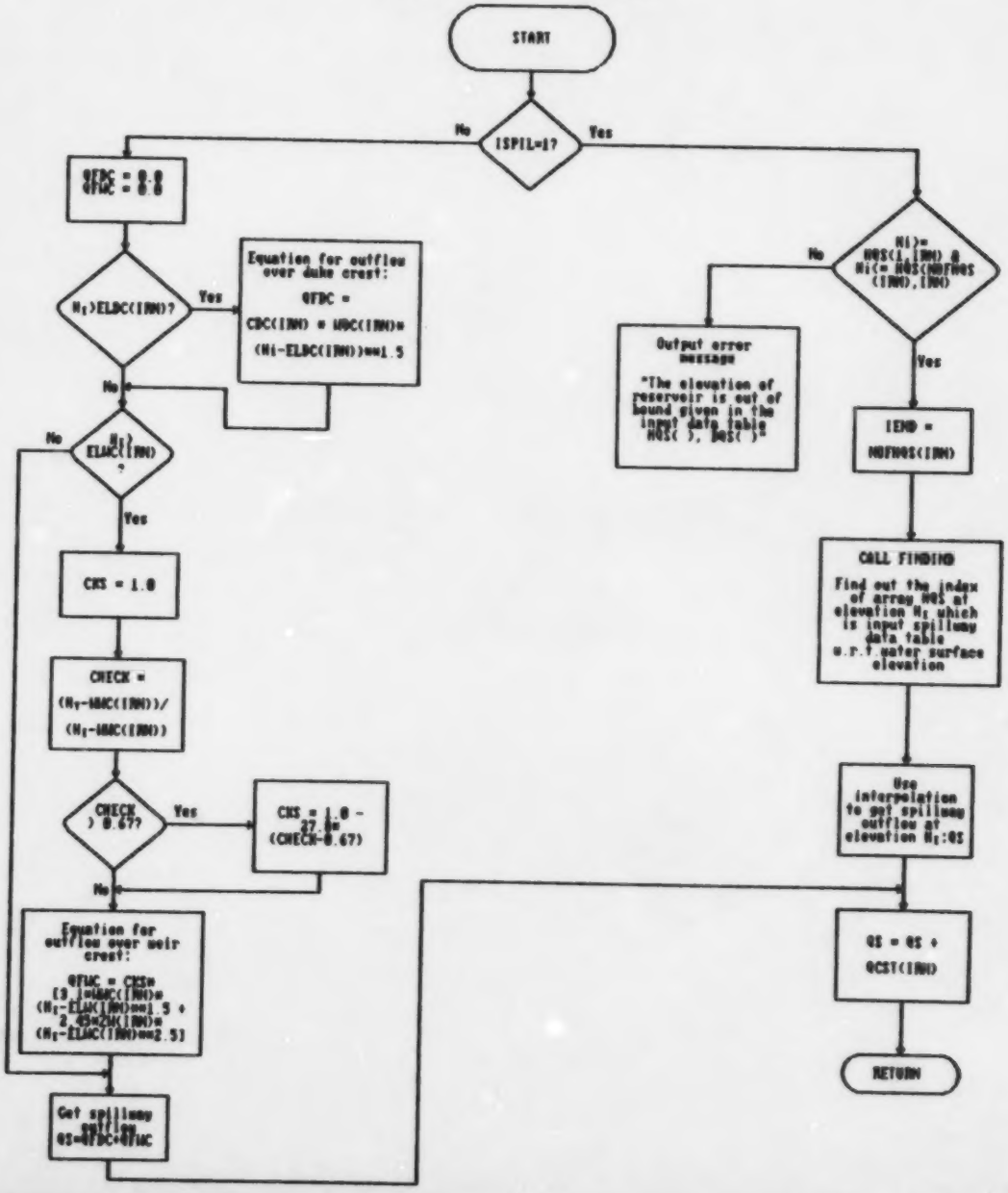
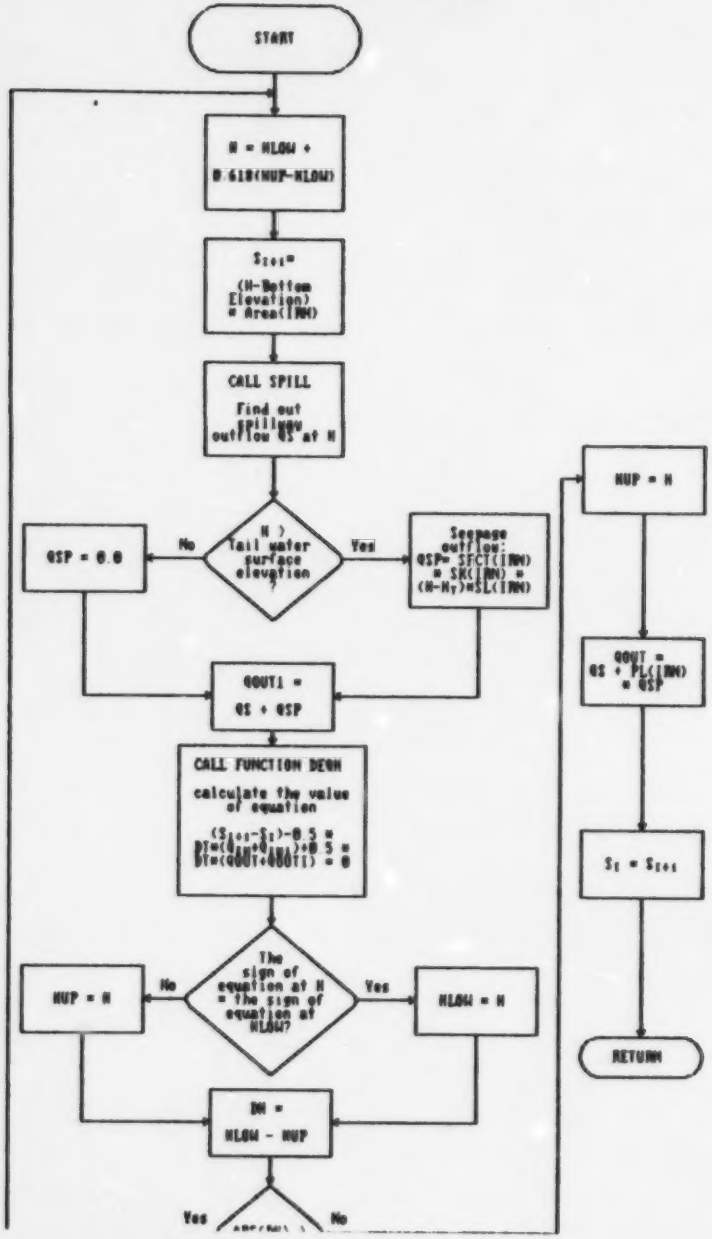


Figure 3.5 (7/7)
Flow Chart - BINARY



3.2 INPUT DATA REQUIREMENTS

The model input data is divided into data that is common and data that is specific to each reservoir or cell.

System Data File

1. System unit (English or Metric) - E
2. Total number of reservoirs in cascade - NR
3. Start time of reservoir routing period - TO
4. Ending time of reservoir routing period - TEND
5. Computation time step - DT
6. Criterion of convergence - ICRI

   ICRI=0 using water surface elevation as convergence criterion

   ICRI=1 using reservoir storage as convergence criterion

7. Spillway option - ISPIL

   ISPIL=1 using input data table

   ISPIL=2 calculate from two equations (weir and crest overflow)

8. Iteration tolerance - EPS
9. Tailwater elevation of the last reservoir - HTL
10. Evaporation

    - if a single value for total simulation period is used - EVAP
    - if a series of evaporation values is used - EVAP = -0.01

11. Inflow into the upstream reservoir (two arrays) -TUP(), QUP()

12. Base series runoff and evaporation three arrays where a series of changing evaporation values are used. If EVAP is not set to -0.01 EVBS() will not be read. -TBS(), QBS(), EVBS()

Reservoir Data File

1. Initial water surface elevation - HO
2. Top of tailings level - HBT
3. Reservoir storage functions - MS

   MS=1 storage is a function of surface area depth

   MS=2 storage is a function of surface elev.

4. Surface area of reservoir - S
5. Void ratio (porosity) of tailings deposits - VR
6. Total length of internal and external dykes - SL
7. Shape factor in seepage equation - SFCT
8. Tailings permeability in seepage equation - SK
9. Fraction of calculated seepage that enters next cell - FS
10. Constant low flow outlet (LFOW) flow - QCST
11. Dyke crest equation - coefficient of discharge - CDC
12. Dyke crest overflow length - WDC
13. Bottom width of weir crest - WWC
14. Elevation of top of dyke - ELDC
15. Elevation of weir invert - ELWC

16. Weir side slope (H/V) - ZW

17. Permeability of weir protection rip-rap - KG

18. Depth of weir sub-base - DRR

19. Fraction of base Q for local runoff - PBS

Notes:

Reservoir storage volume relationship may be entered as a table.

Spillway rating curve may be entered as a table.

3.3 INPUT DATA - QUIRKE TAILINGS BASIN

In modelling the Quirke tailings basin, only the five cell basin is considered. The upstream Gravel Pit Lake is assumed to be completely isolated, and no overtopping will occur during very extreme flood events. This assumption is confirmed by a separate analysis.

Under low flow conditions modelling of the five cell system may be done with or without a controlled (fixed) inflow from Gravel Pit Lake.

The base inflow series is based on the runoff into Gravel Pit Lake.

System Data

The common input data for the five cell system is as follows:

NR - Number of reservoirs in cascade - 5

DT - Computation time step 3 hr for flood routing
1 day for low flows

ISPIL - Spillway discharges using formulae

HTL - Tailwater below Dam G2 - 1260 feet

Cell Data

Input data of the five cells is given on the table overleaf.

TABLE 3.1

QUIRKE TAILINGS BASIN - INPUT DATA

CELL ID		14	15	16	17	18
Kn	Reservoir No.	1	2	3	4	5
Ho	Initial Water Level - Ft.	1310	1297	1283	1270	1268
HBt	Top of Tailings - Ft.	1308	1295	1281	1268	1266
S	Area of Water Surface - $Ft^2 \times 10^6$	6.663	3.300	5.449	1.230	1.628
VR	Void Ratio (Porosity) Tailings	0.5	0.5	0.5	0.5	0.5
SL	Total Length of Dams - Ft.					
	Internal	3850	3800	1500	1600	-
	External	1200	750	1500	-	1200
SFCT	Seepage Factor	0.6	0.6	0.6	0.6	0.6
SK	K in Seepage Eq x 10^{-5} Ft./sec. (average)	1.64	1.64	1.64	1.64	1.64
FS	Fraction of Seepage to Next Cell	0.70	0.92	0.66	1.00	0.00
QCST	LFOW-Q - Cfs (to be selected)					0
CDC	Discharge Coefficient for Dyke Crest	3.1	3.1	3.1	3.1	3.1
WDC	Length of Overflow Dyke Crest - Ft.	3780	3730	1430	1530	1130
WWC	Bottom Width of Weir - Ft.	20	20	20	20	60
ELDC	Dyke Crest Elevation - Ft.	1314	1301	1287	1274	1274
ELWC	Weir Invert Elevation - Ft.	1311.5	1298.5	1284.5	1271.5	1268.5
ZW	Weir Side Slope	10	10	10	10	2
KG	Rip-Rap Permeability - Ft./sec.	2	2	2	2	2
DRR	Thickness of Weir Sub-base - Ft.	1.5	1.5	1.5	1.5	1.5
PBS	% Local Runoff	0.177	0.068	0.162	0.030	0.081

3.4 INPUT DATA - PANEL TAILINGS BASIN

The Panel mine tailing complex consists of two cells, Strike Lake and the South Basin.

It is assumed that the existing main dyke and central dyke in Strike Lake will not act as continuous internal barriers.

Excess flood flows will be discharged into the South Basin through an emergency spillway 1200 feet east of Dam D. Excess flood flows from the South Basin will be discharged over the emergency weir (Dam F) at the south west end of the basin.

The base inflow series is based on the total area of the Strike Lake and South Basin sub-catchments.

System Data

NR - Number of reservoirs in cascade - 2

DT - Computation time step 3 hr for flood routing
1 day for low flows

ISPIL - Spillway discharges using formulae

HTL - Tailwater below Dam F - 1315 feet

Cell Data

Input data of the two cells is given in the table overleaf.

TABLE 3.2

PANEL TAILINGS BASIN - INPUT DATA

CELL ID		STRIKE LAKE	SOUTH BASIN
Rn	Reservoir No.	1	2
Ho	Initial Water Level - Ft.	3360	3316
HBt	Top of Tailings - Ft.	3358	3314
S	Area of Water Surface - $Ft^2 \times 10^6$	7.309	6.424
VR	Void Ratio (Porosity) Tailings	0.5	0.5
SL	Total Length of Dams - Ft.		
	Internal	0	0
	External	1600	950
SFCT	Seepage Factor	0.6	0.6
SK	K in Seepage Eq x 10^{-6} Ft./Sec.	1.64	1.64 (average)
FS	Fraction of Seepage to Next Cell	0	0
QCST	LFOW-Q - Cfs	0	0
CDC	Discharge Coefficient for Dyke Crest	3.1	3.1
WDC	Length of Overflow Dyke Crest - Ft.	580	0
WWC	Bottom Width of Weir - Ft.	20	25' + 10'
ELDC	Dyke Crest Elevation - Ft.	3365	3322
ELWC	Weir Invert Elevation - Ft.	3360	3319 (W = 25') 3316 (W = 10')
ZW	Weir Side Slope	0	0
KG	Rip-rap Permeability - Ft./Sec.	0	0
DRR	Thickness of Weir Sub-base - Ft.	0	0
PBS	% Local Runoff	.602	.398

4. HYDROLOGY AND HYDROGEOLOGY

4.1 GENERAL

The Quirke and Panel tailings basins are located in the Serpent River catchment and measure 2.32 square miles and 1.08 square miles, respectively. While the drainage area of the Serpent River is about 541 square miles, those of Gravel Pit Lake (Quirke Lake) and the Strike Lake - South Tailings areas are therefore very small in comparison. The sizes of these drainage areas require that the special characteristics of "small drainage basins" must be constantly borne in mind in evaluating the hydrology of the study area.

Of relevance to the flood analyses of the Quirke and Panel areas is the fact that the Serpent River drainage basin is adjacent to and south-east of the Mississagi drainage basin, Figure 4.1. The Mississagi basin was the subject of a recent detailed Probable Maximum Precipitation (PMP) study for Ontario Hydro, and, with appropriate corrections, the results of that study could be applied to the study area.

Topographically, the landform is the product of glacial action. The punctuation of the ground by lakes and depressions have their effect in providing storage and thus reducing the flood peaks. The percentage of lakes in the Quirke drainage basin and in the intermediate catchment between Dunlop Lake and Quirke Lake was found to be 10.5% and 13.6%, respectively. Although the study area is small, when the extent of large rain clouds is taken into consideration, then the rising landform, from North Channel and proceeding inland, must be recognized, i.e., the orographic effects of rainfall must be taken into account.

Figure 4.1

Relative Location of Mississagi River and Serpent River Basins

4.2 DATA BASE

4.2.1 Streamflow

Data for the stream gauging stations which are listed in Appendix D was examined in this study. However, in terms of securing data which was natural and free from the effects of regulation or storage, and which came from as small a drainage area as possible, it was decided to use only two stations. These are:

No.	Name	Area	Period
02CD006:	Serpent River above Quirke Lake:	157 km^2	(1967-88)
02CD002:	Serpent River at Dunlop Lake:	109 km^2	(1977-88)

A double mass curve of the recorded data over corresponding periods (1978-88) is shown in Figure 4.2. The almost straight line of the graph indicates that the quality of the data is high. Consequently, it was decided to attempt to extend the data at Dunlop Lake back to 1967/68, from April to October, on a monthly basis. The extension was in the form:

$$Q_{002} = A_* Q_{006} + B$$

where:

Q_{002} = flow at station 02CD002
Q_{006} = flow at station 02CD006
A, B = coefficients.

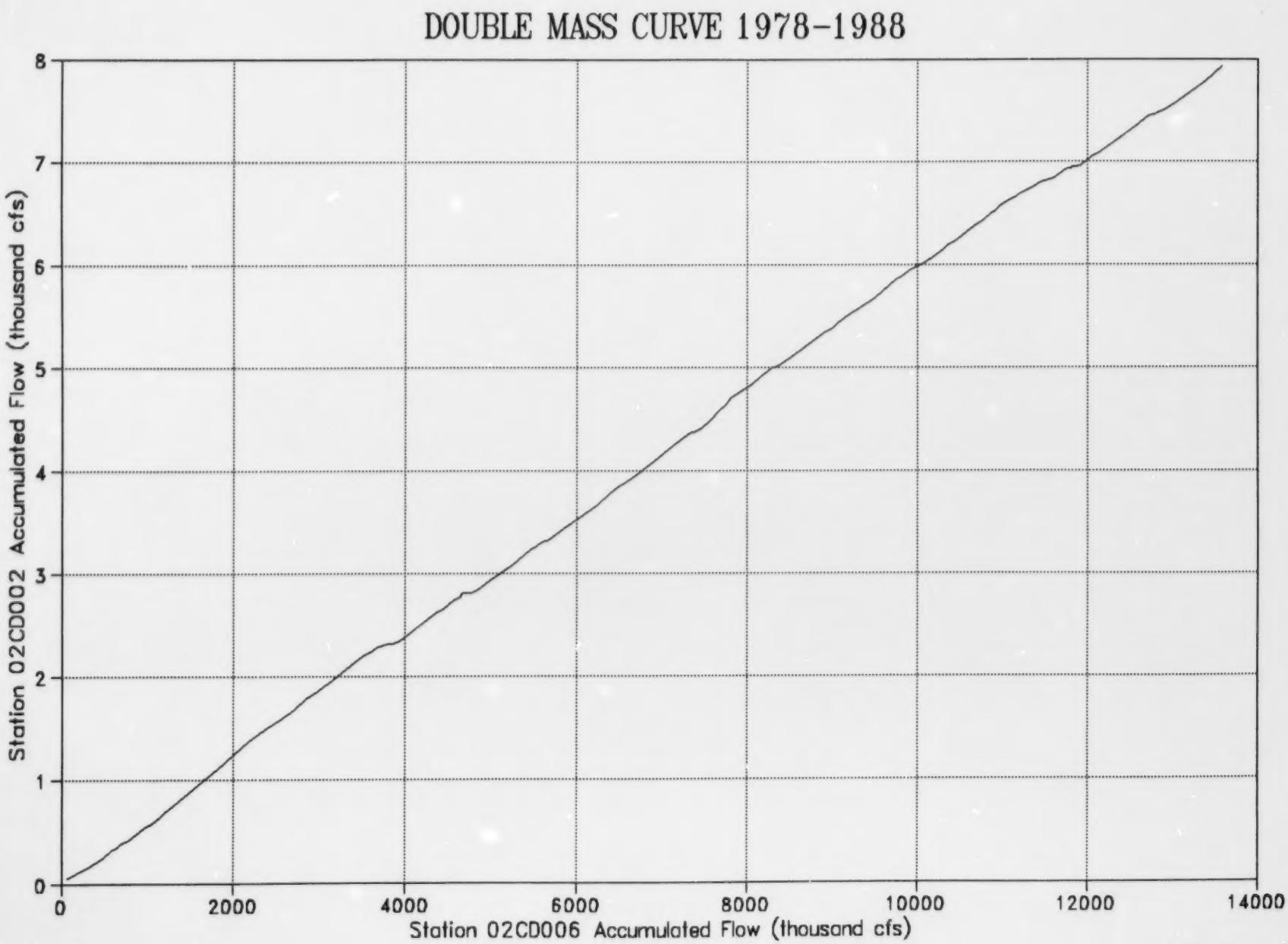

DOUBLE MASS CURVE 1978-1988
Station 02CD002 Accumulated Flow (thousand cfs)
0
1
2
3
4
5
6
7
8
0
2000
4000
6000
8000
10000
12000
14000
Station 02CD006 Accumulated Flow (thousand cfs)

FIGURE 4.2

The coefficients and the coefficients of determination of the equation, r^2, are given in Table 4.1 below. From the values of r^2 which have been obtained, it can be seen that only the month of September shows the value of r^2 to be slightly less than 0.8.

TABLE 4.1

REGRESSION COEFFICIENTS

	Average Monthly Flows			Daily Flows		
	A	B	r^2	A	B	r^2
Apr.	0.729	-1.017	0.93	0.656	-0.473	0.87
May	0.789	-0.647	0.97	0.780	-0.604	0.95
June	0.717	-0.256	0.99	0.730	-0.286	0.96
July	0.815	-0.131	0.96	0.848	-0.366	0.92
Aug.	0.706	-0.273	0.91	0.646	-0.200	0.81
Sept.	0.576	-0.259	0.81	0.604	-0.292	0.77
Oct.	0.722	-0.626	0.98	0.685	-0.515	0.92

The high values of "r^2" of these regression equations justified the extension of the data for Station 02CD002 back to 1967. It was then possible to obtain flows for the intermediate catchment between these two gauging stations by subtracting the flows at 02CD002 from those at 02CD006. The area of this intermediate catchment is 18.53 square miles, and both the Quirke and Panel drainage areas lie within it.

The flow data (02CD006-02CD002) was used to obtain, on a pro rata basis, the flow and flood characteristics of the Quirke and Panel areas.

4.2.2 Drainage Areas

The pertinent drainage areas relating to both the Quirke and Panel areas are as follows:

TABLE 4.2

DRAINAGE AREAS

Name	Area Sq. Miles
Gravel Pit Lake	2.32
Cell 14	0.41
Cell 15	0.16
Cell 16	0.38
Cell 17	0.07
Cell 18	0.19
Frayn Lake	0.38
Strike Lake	0.60
South Tailings Area	0.40

4.2.3 Precipitation

The average annual precipitation on the study area was taken to be that at Elliot Lake, the nearest meteorological station. This station is about 10 miles from the centre of the study area, and the mean recorded annual precipitation (1951-80) there is as follows:

TABLE 4.3

PRECIPITATION AT ELLIOT LAKE

Mean annual rainfall	703.2 mm
Mean annual snowfall, water equivalent	219.0 mm
Mean annual total precipitation,	926.3 mm

4.2.4 Probable Maximum Precipitation (PMP)

A specific study to determine the PMP at the study area was not carried out. Rather, the detailed study which was done by M.M. Dillon for Ontario Hydro, listed in Annex A, Section B, was used as the basis for determining the PMP on the study area.

The Dillon study provided both spring and summer depth - area - duration (D-A-D) curves of the PMP; these curves take the orographic influence of the land into account and were derived for periods ranging from 6 hours to 96 hours, and for drainage areas ranging from 1 000 km^2 to 10 000 km^2. Since it is known that the depth increases with decreasing drainage area, it was therefore necessary to determine the corresponding D-A-D values for the very much smaller areas at Quirke and Panel.

In order to perform this extrapolation, the generally accepted notion that point rainfall can be taken to be the same as rainfall over a 25 km^2 (9.65 sq. miles) area was acknowledged. This notion together with the World Meteorological Organisation (WMO) guidelines for reducing point rainfall to rainfall covering 1000 km^2 were applied to the Dillon curves. Figures 4.3 and 4.4 and Table 4.4 show the D-A-D curves and specific values for spring and summer, respectively. This data clearly verifies the

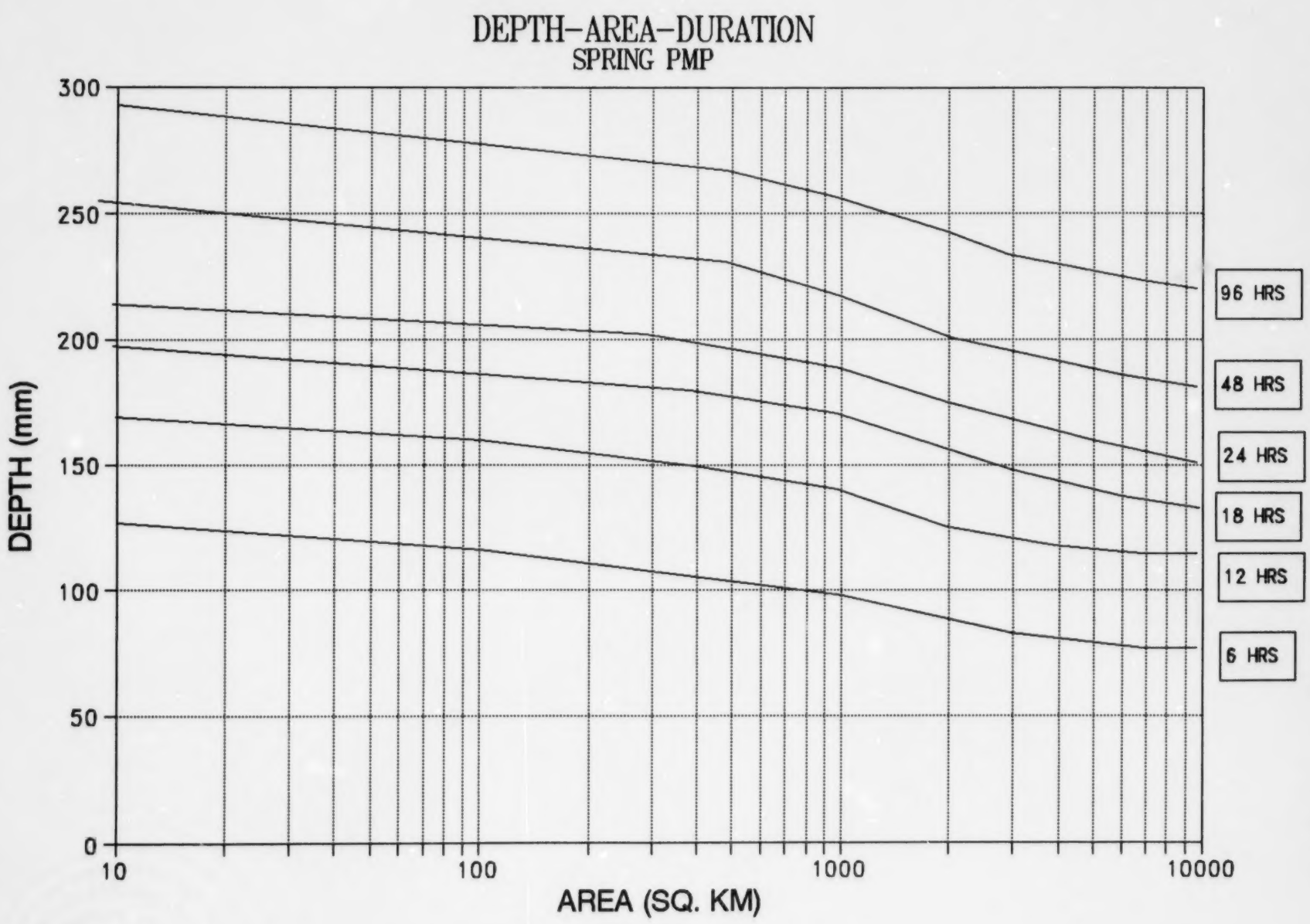

DEPTH-AREA-DURATION
SPRING PMP
300
250
200
150
100
50
0
DEPTH (mm)
10
100
1000
10000
AREA (SQ. KM)
96 HRS
48 HRS
24 HRS
18 HRS
12 HRS
6 HRS

FIGURE 4.3

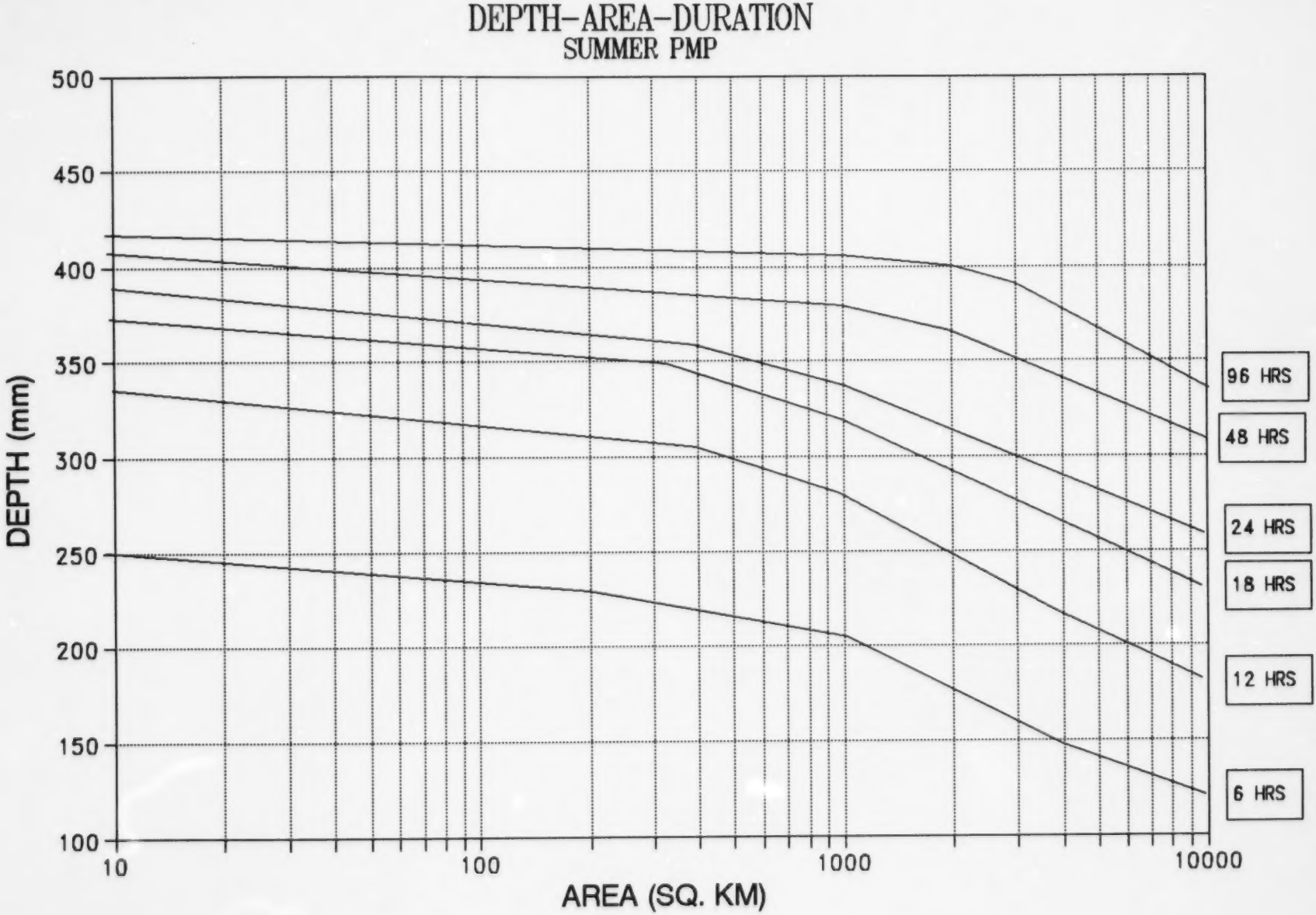

DEPTH-AREA-DURATION
SUMMER PMP
500
450
400
350
300
250
200
150
100
DEPTH (mm)
10
100
1000
10000
AREA (SQ. KM)
96 HRS
48 HRS
24 HRS
18 HRS
12 HRS
6 HRS

FIGURE 4.4

fact that the PMP varies seasonally, i.e. the spring PMP should not be expected to be same as that for the summer.

TABLE 4.4

PROBABLE MAXIMUM DEPTH

Duration (hours)	Spring (mm)	Summer (mm)
6	124	253
12	167	335
18	193	374
24	211	392
30	224	397
36	233	401
42	241	405
48	248	407
54	255	408
60	263	409
66	270	410
72	274	411
78	278	412
84	282	413
90	286	414
96	289	415

4.2.5 Snowmelt

In order to take account of snowmelt in determining the Probable Maximum Flood (PMP) which would occur in the spring, both the depth of the snowpack and the critical temperature sequence must be established. The Dillon Study provided the 1:100 year and 1:500 year water equivalent snowpack values, and of these the 1:100 year

values were used in this study. The values for Red Rock Falls at the southern end of the Mississagi basin were taken as being indicative of the snowpack on the study area: Table 4.5 below gives these snowpack values for the frequency distributions which were considered.

TABLE 4.5

1:100 YEAR SNOWPACK AT RED ROCK FALLS

	Frequency Distribution			
	Normal	Gumbel Extreme Value	3 Parameter Log-Normal	Log-Pearson Type III
Snow pack (mm)	275	255	267	254

The value of 10.51 inches (267 mm) was adopted in this study.

A critical temperature regime covering the period April 11 - May 4 was also provided in the Dillon study for the Mississagi basin. These values are shown in Figure 4.5.

When the Dillon temperature regime as shown in Figure 4.5 was used, the snowpack disappeared in 10 days. However, a greater flood peak was obtained by using the regime labelled "adopted sequence" in Figure 4.5; under this regime, the snowpack disappeared in six days.

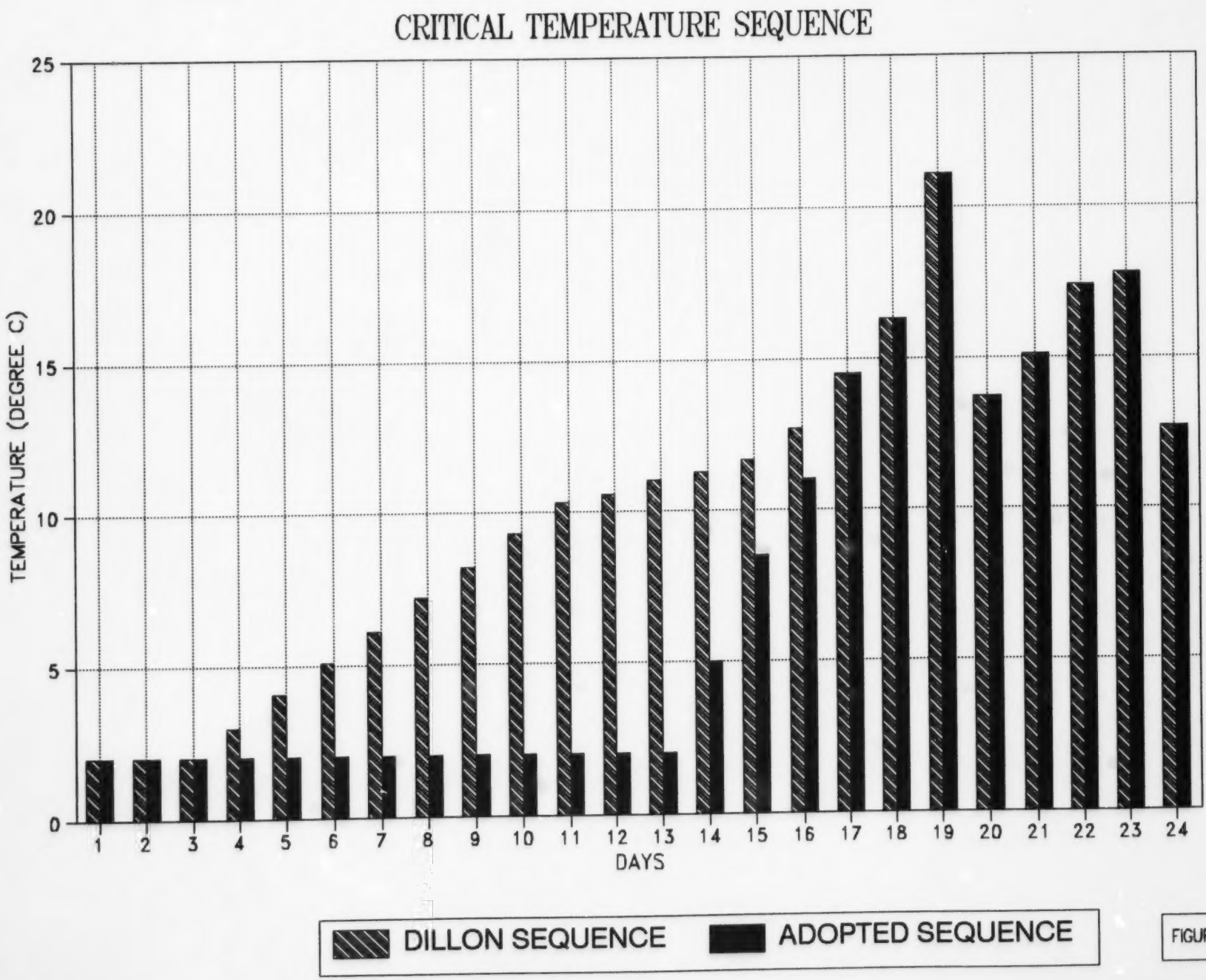

CRITICAL TEMPERATURE SEQUENCE
TEMPERATURE (DEGREE C)
0
5
10
15
20
25
1 2 3 4 5 6 7 8 9 10 11 12 13 14 15 16 17 18 19 20 21 22 23 24
DAYS
DILLON SEQUENCE
ADOPTED SEQUENCE

FIGURE 4.5

TABLE 4.6

ADOPTED CRITICAL TEMPERATURE SEQUENCE

Day	Temperature	
	°C	°F
1	2.0	35.6
2	5.0	41.0
3	8.5	47.3
4	11.0	51.8
5	14.5	58.1
6	16.3	61.3
7	21.1	70.0

For the calculation of the snow melt, the following equation was used.

$M = 0.06\ (T - 24), \qquad 34°F < T < 66°F$

where,

M = snowmelt in inches per day

T = temperature in °F

0.06 = appropriately chosen factor depending on the basin characteristic.

4.3 FLOWS

The flow data which was generated for the intermediate catchment between the stream gauging stations 02CD002 and 02CD006 was used to obtain both minimum flow and average low flow sequences, on a pro rata area basis, for both the Quirke and Panel drainage area.

The average low flows were taken as the lowest average flow values over consecutive 30, 60, 90 and 120 days in each year. Through the use of suitable frequency distributions, it was then possible to ascertain the return period of these average low flows. The results of the computer analyses are shown in Figures 4.6 and 4.7 for the Quirke and Panel drainage areas, respectively.

For the determination of the minimum flows only the period May to October of each year was considered since during this period the runoff values are at their lowest.

The minimum monthly average flows for the intermediate catchment are shown in Figure 4.8. The corresponding flows which would occur at the Quirke and Panel areas are, respectively, 12.5% and 5.4% of the values shown in Figure 4.8.

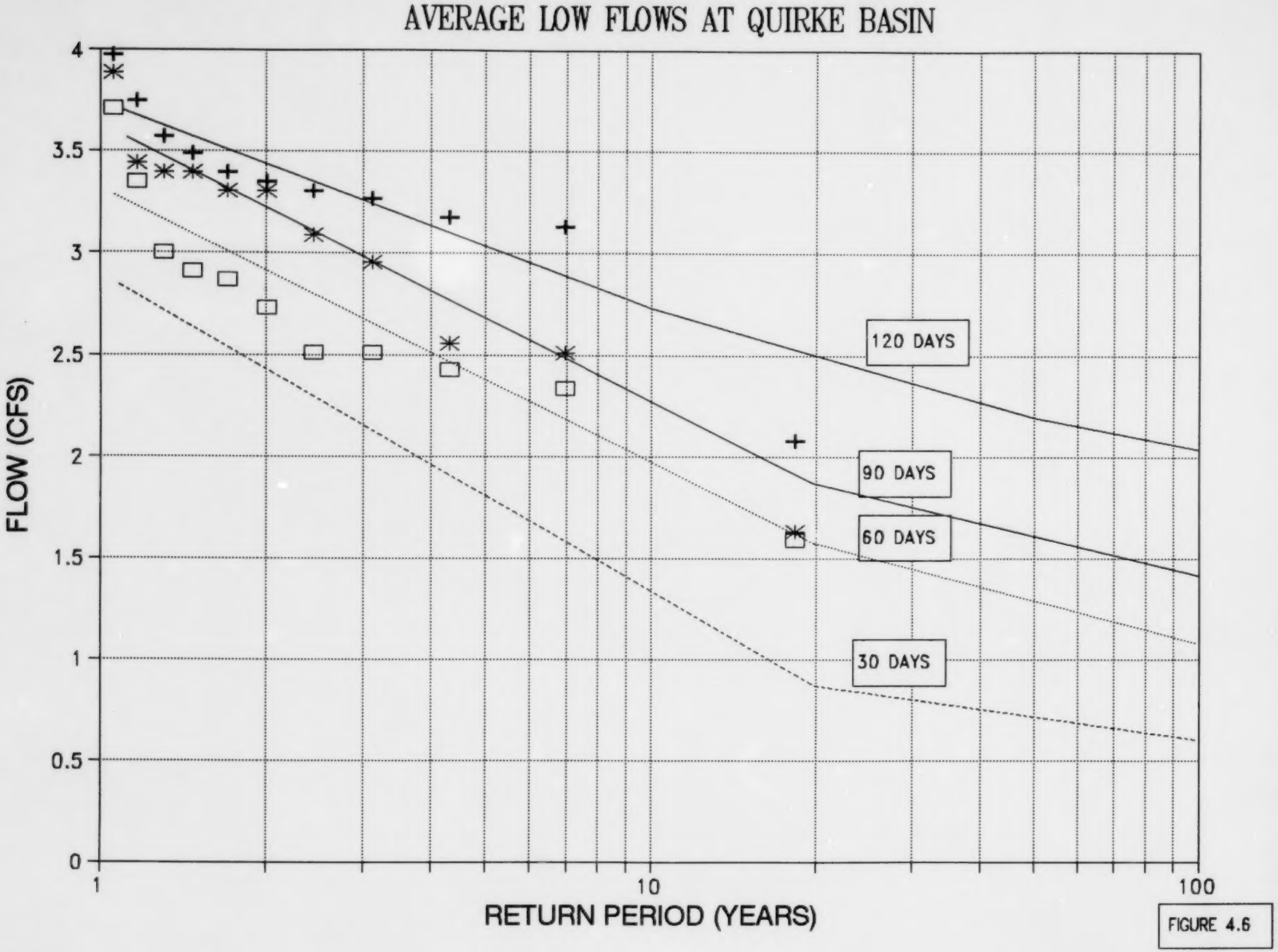

AVERAGE LOW FLOWS AT QUIRKE BASIN
FLOW (CFS)
4
3.5
3
2.5
2
1.5
1
0.5
0
1
10
100
RETURN PERIOD (YEARS)
120 DAYS
90 DAYS
60 DAYS
30 DAYS

FIGURE 4.6

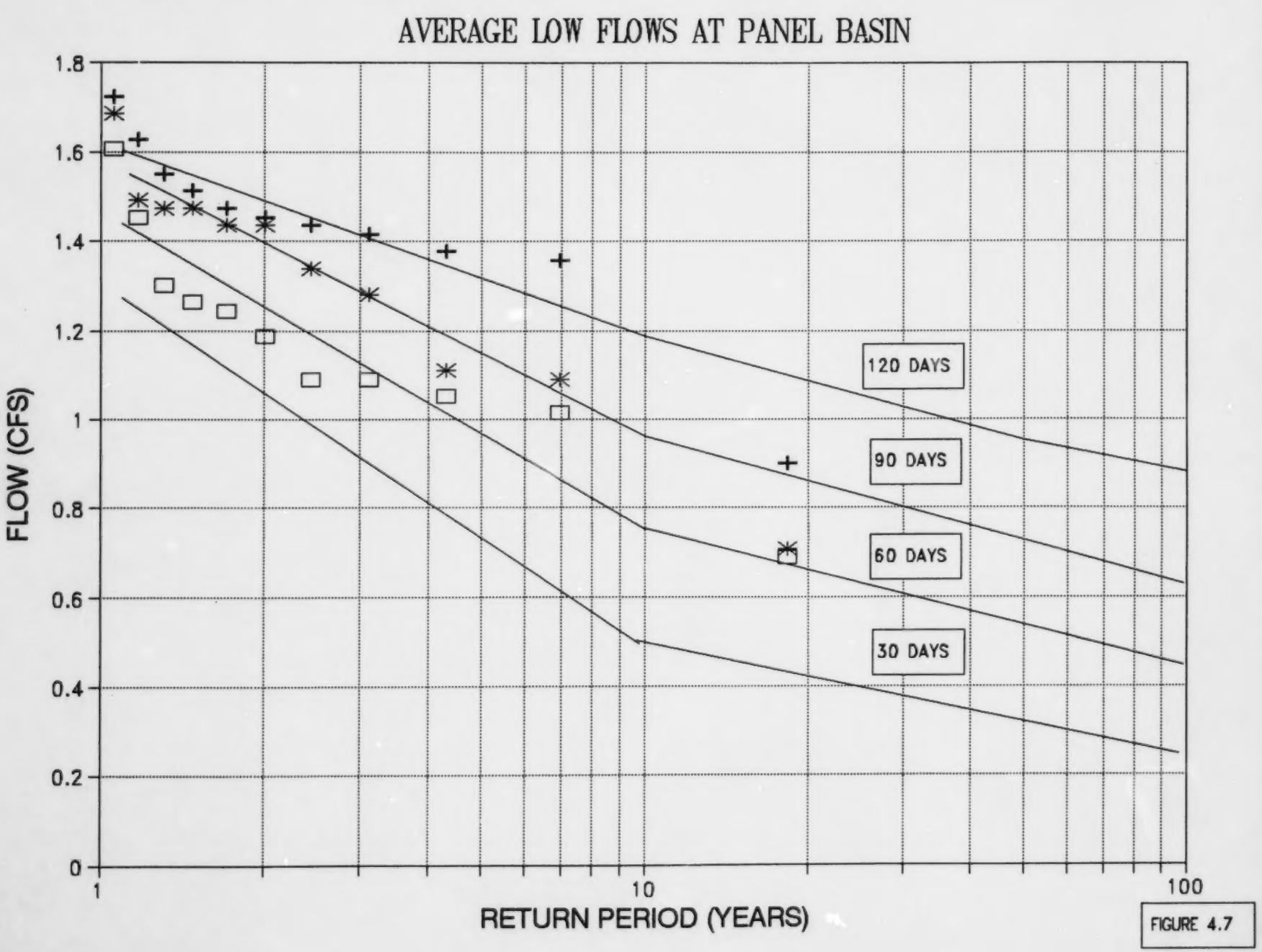

AVERAGE LOW FLOWS AT PANEL BASIN
FLOW (CFS)
1.8
1.6
1.4
1.2
1
0.8
0.6
0.4
0.2
0
1
10
100
RETURN PERIOD (YEARS)
120 DAYS
90 DAYS
60 DAYS
30 DAYS

FIGURE 4.7

MINIMUM MONTHLY FLOWS
(02CD006-02CD002)

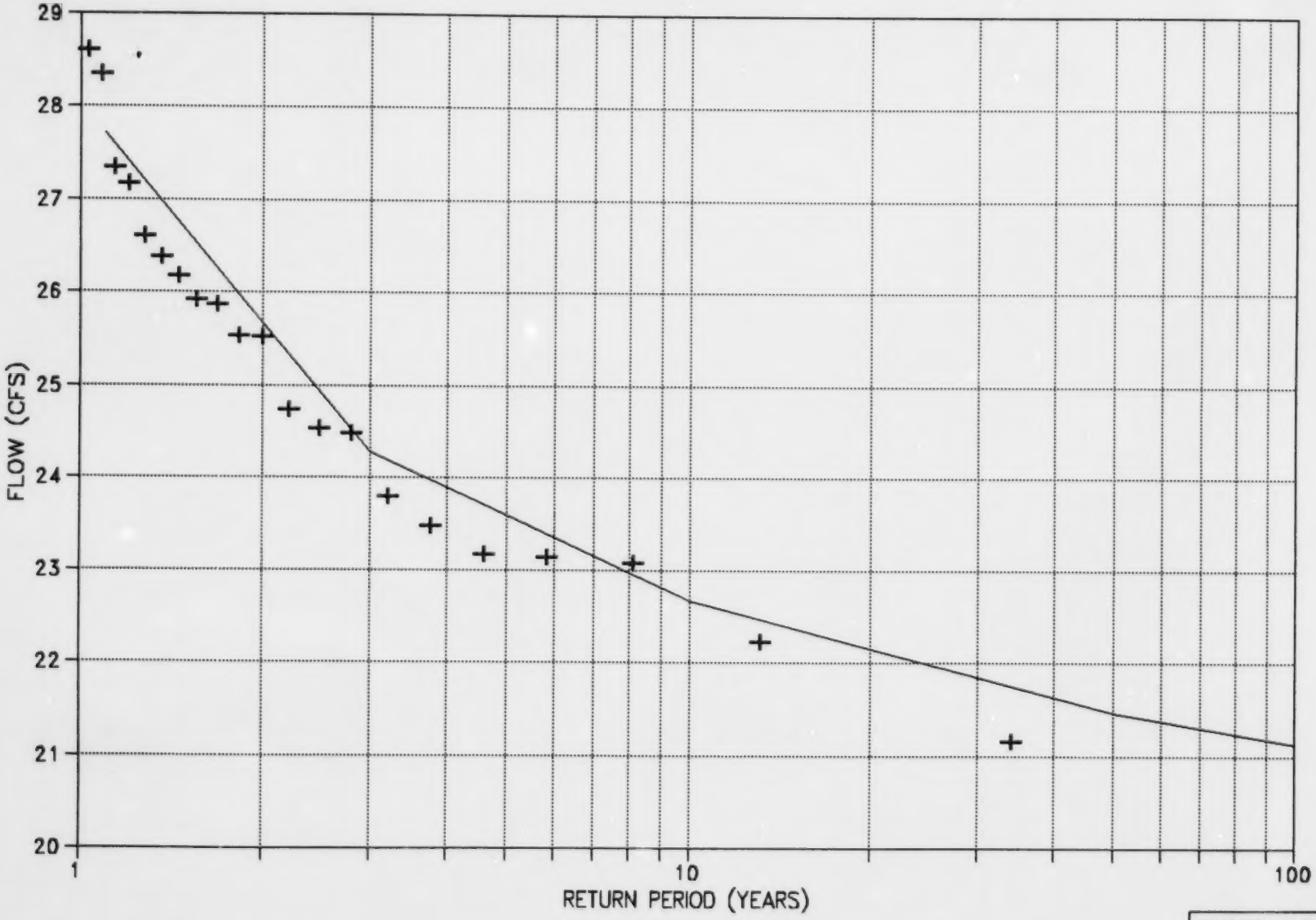


FIGURE 4.8

4.4 FLOODS

4.4.1 Extreme Floods

The recorded streamflow data for the intermediate catchment, given by the difference between the flows at stations 02CD006 and 02CD002, was used to obtain a yearly maximum flow series for both spring and summer floods. Analysis of the data using standard frequency distributions allowed floods of various return periods to be determined.

The frequency distributions which were selected were log Pearson Type III, log-normal, three parameter log-normal, extreme value (Gumbel Type I). However, an examination of the standard error of each distribution indicated that only two distributions gave acceptable estimates of the floods: these distributions were log-Pearson Type III for the spring floods and log-normal for the summer floods. Figures 4.9 and 4.10 show these flows for spring and summer in the intermediate catchment.

Since the Quirke and Panel drainage areas are sub-basins of the intermediate catchment, the extreme flood events for these sub-basins were obtained by empirical formulae of the form:

$$\text{Flow} = f\ (\text{Area})^n$$

where $0.75 < n < 0.83$ for small basins.

If the flood events of the sub-basins were obtained by first pro rating the flows from the intermediate catchment on an area basis, and then subjecting this data to a frequency analysis, then the resulting floods would

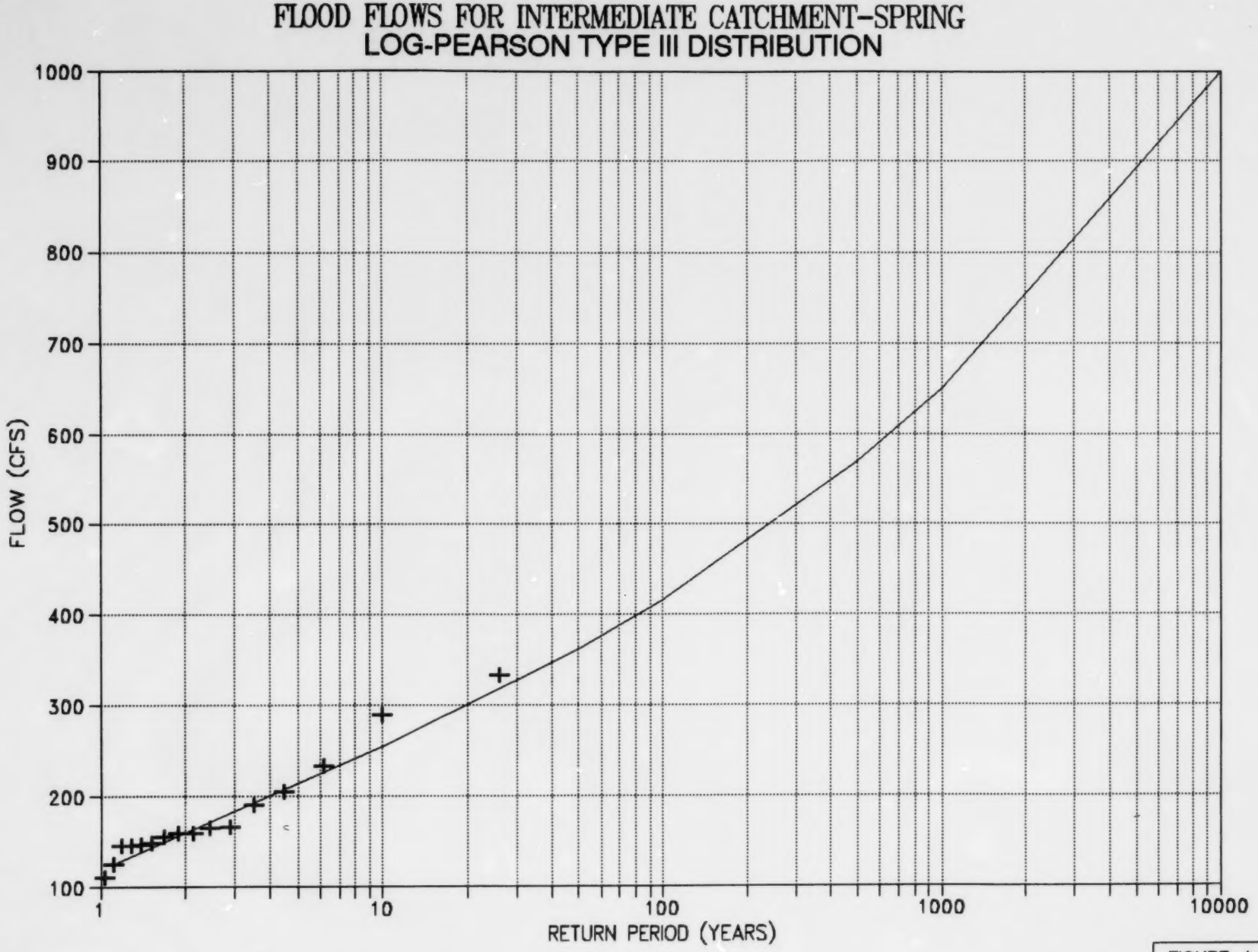

FLOOD FLOWS FOR INTERMEDIATE CATCHMENT-SPRING
LOG-PEARSON TYPE III DISTRIBUTION
FLOW (CFS)
1000
900
800
700
600
500
400
300
200
100
1
10
100
1000
10000
RETURN PERIOD (YEARS)

FIGURE 4.9

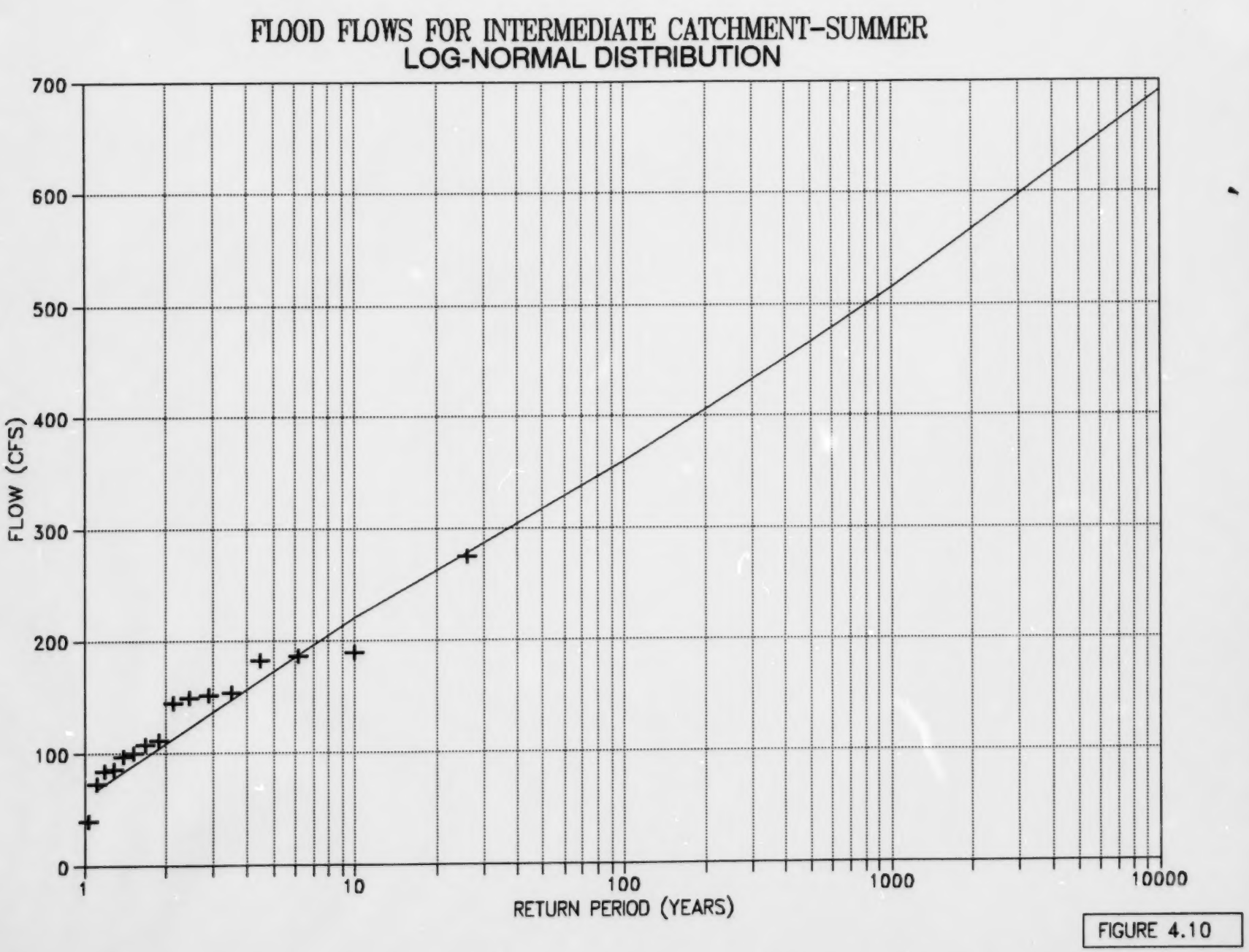

FLOOD FLOWS FOR INTERMEDIATE CATCHMENT-SUMMER
LOG-NORMAL DISTRIBUTION
FLOW (CFS)
700
600
500
400
300
200
100
0
1
10
100
1000
10000
RETURN PERIOD (YEARS)

FIGURE 4.10

be more than the corresponding floods obtained by using the empirical formulae: this is because "n", in the formula quoted above would be implicitly equal to 1.0.

The estimated flood flows for the Quirke and Panel basins, based on n = 0.75 and on the adopted floods for the intermediate catchment, are given in Table 4.7 below. The table also shows, for the drainage areas of the Quirke and Panel basins, the floods which would be obtained from the envelope of the average daily 1:10 000 year floods for six gauging stations in the area: the drainage areas at these stations varies from 40.6 km^2 to 1350 km^2. No attempt was made, in obtaining this envelope, to differentiate between spring and summer floods; only the largest annual floods were used. The consistency between the 1:10 000 year flood values is clearly demonstrated.

TABLE 4.7

EXTREME FLOOD EVENTS

	FLOWS CFS		
Basin	1:100	1:10 000	
		n = 0.75	Envelope
Quirke			
- spring	87	210	170
- summer	76	145	
Panel			
- spring	47	112	80
- summer	40	77	

Research into the available streamflow data for Ontario indicated that the ratio of instantaneous peak to the average daily peak could vary from 1.08 to 3.21. For the purposes of this study a value of 1.7 was used.

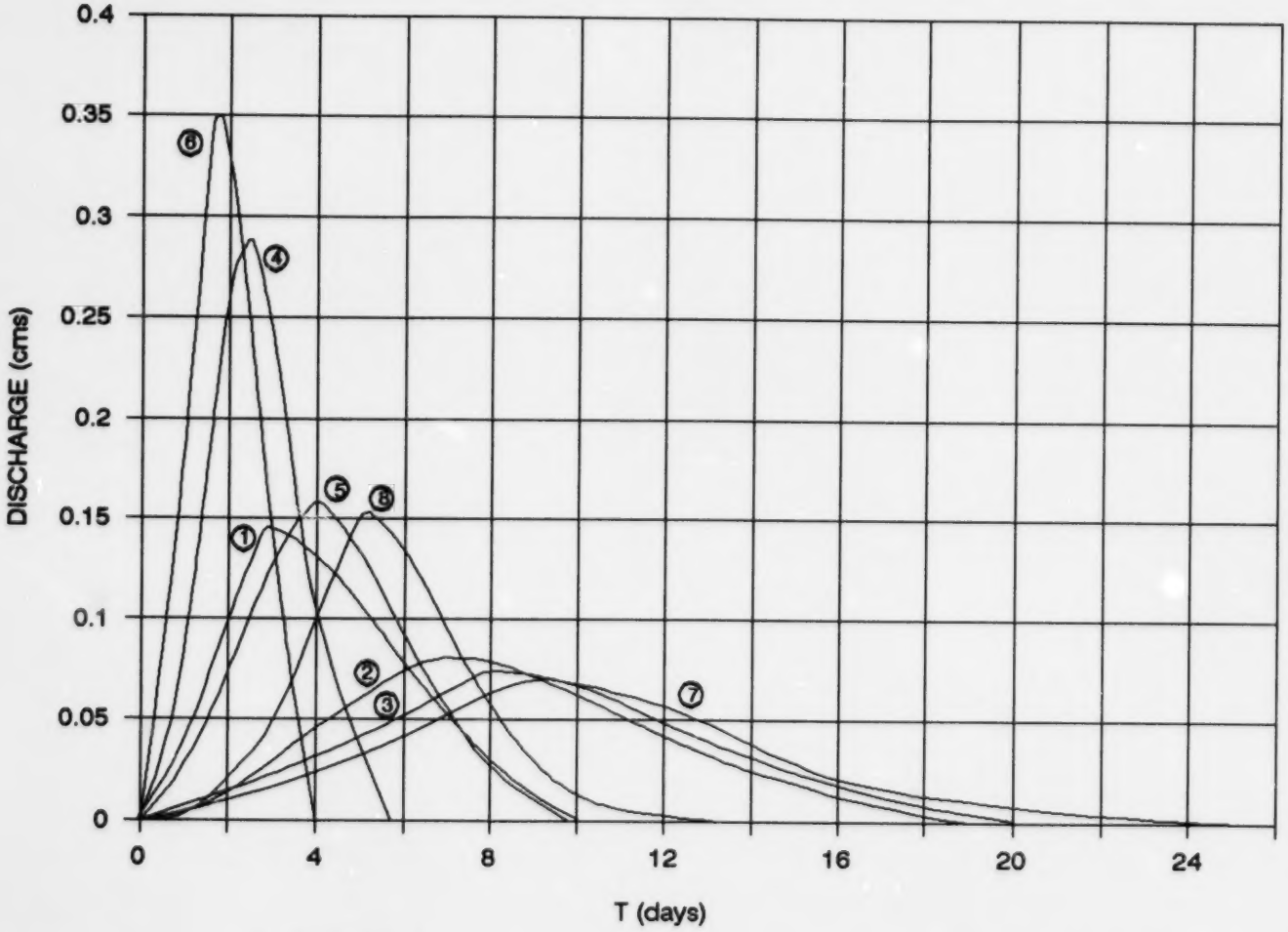
4.4.2 Unit Hydrographs

An examination of the flow records for the intermediate catchment allowed six floods or high flows to be identified. From these six hydrographs, unit hydrographs for the intermediate catchment were prepared. Similarly, two unit hydrographs for station 02CF100, Vermilion River near Capreol (245 square miles) were also prepared to have a comparison with a larger drainage basin in the general geographic area. These unit hydrographs are shown in Figure 4.11.

Since the Quirke and Panel areas are sub-basins of the intermediate catchment, it seemed appropriate to use the same basic unit hydrograph of the intermediate catchment for the sub-basins. The unit hydrograph No. 6, Figure 4.11, produces the greatest peak discharge.

Since the period of the recorded data is short, it is possible that the flood which would give rise to the most severe unit hydrograph may not have been recorded. It was therefore decided, as a matter of comparison, to prepare synthetic unit hydrographs (Snyder's unit hydrographs): one for the Quirke basin and one for the Panel basin. These 3-hour unit hydrographs were prepared by the use of standard empirical formulae and are shown in Figure 4.12.

UNIT HYDROGRAPHS



Period of Data

① May 29-June 19 (1970)
② Apr 13-May 2 (1976)
③ Mar 11-May11 (1988)
④ Apr 11-Apr 19 (1983)
⑤ Mar 5-Mar 21 (1983)
⑥ May 28- June 9 (1983)
⑦ Apr 10-May 25 (1985)
⑧ Apr 4-Apr 14 (1980)

①, ② 02CF100

③ – ⑧ △ (02CD006-002)

FIGURE 4.11

SELECTED UNIT HYDROGRAPHS - QUIRKE & PANEL BASINS

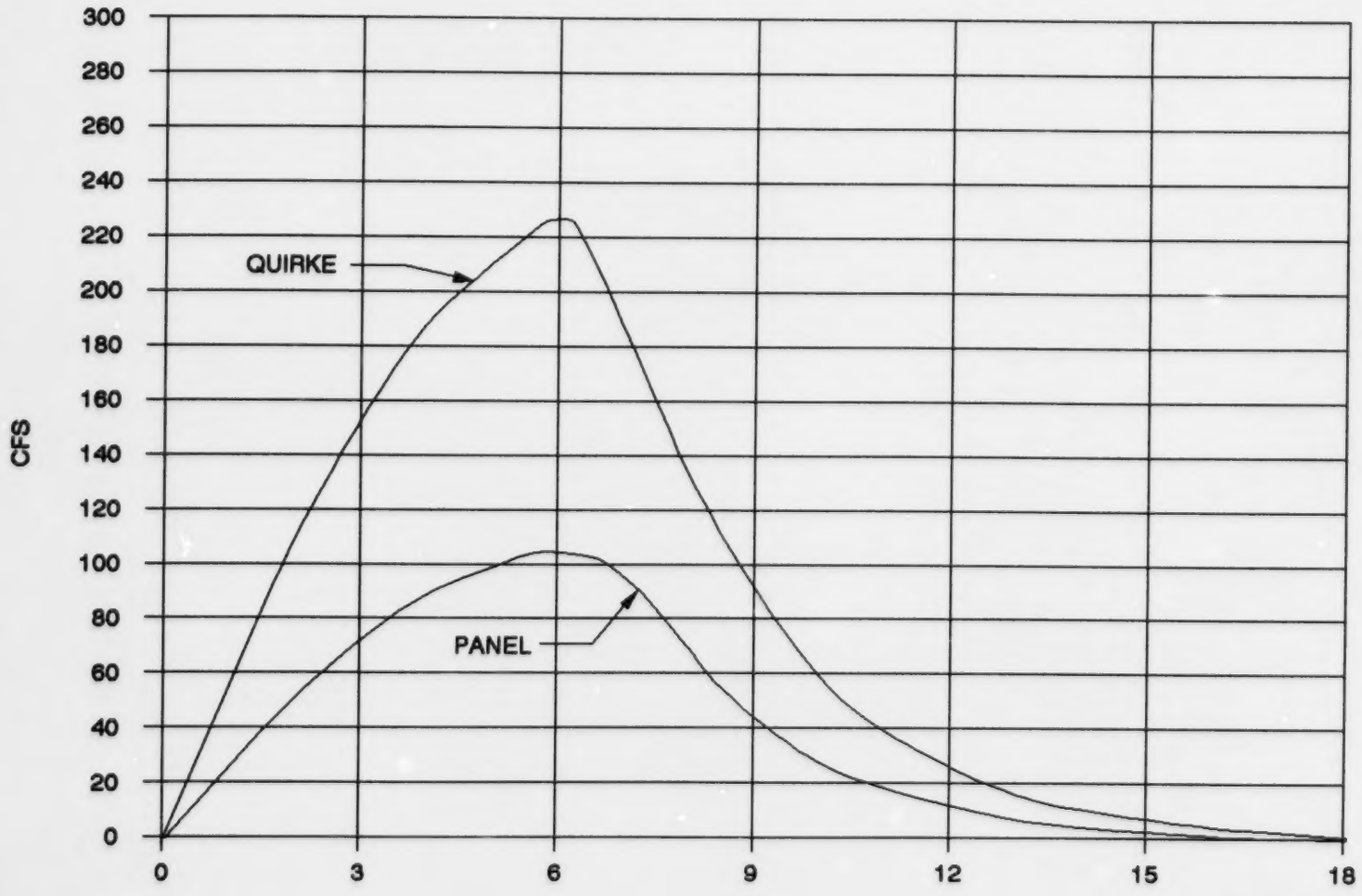


Figure 4.12

4.4.3 Probable Maximum Flood (PMF)

In order to calculate the PMF, the computer program HEC-1, developed by the U.S. Army Corps of Engineers, was used. One of the inputs to this program is basin losses which must be deducted from the PMP to obtain the effective rainfall; this loss was taken as zero to obtain the maximum effect. It is also necessary in using the HEC-1 program to place or order the 3 hourly PMP increments in such a way as to produce the maximum peak outflow. The program also allows for the inclusion of snowmelt which is an obvious element in producing the spring floods.

Figures 4.13 to 4.16 shows the PMP and the PMF for the Quirke and Panel basins for both spring and summer conditions. Table 8 below gives the 3 hour peak values.

TABLE 4.8

3-HOUR PROBABLE MAXIMUM FLOOD PEAKS

	3-Hour Peak Flow (cfs)		Flood Volume (Ac-ft)	
	Spring	Summer	Spring	Summer
Basin				
Quirke	1163	1947	2665	1986
Panel	541	906	1241	925

Since the floods which were produced by using the synthetic unit hydrograph were far more severe than any of those in Figure 4.11, it was decided to use these higher flood values in the simulation studies.

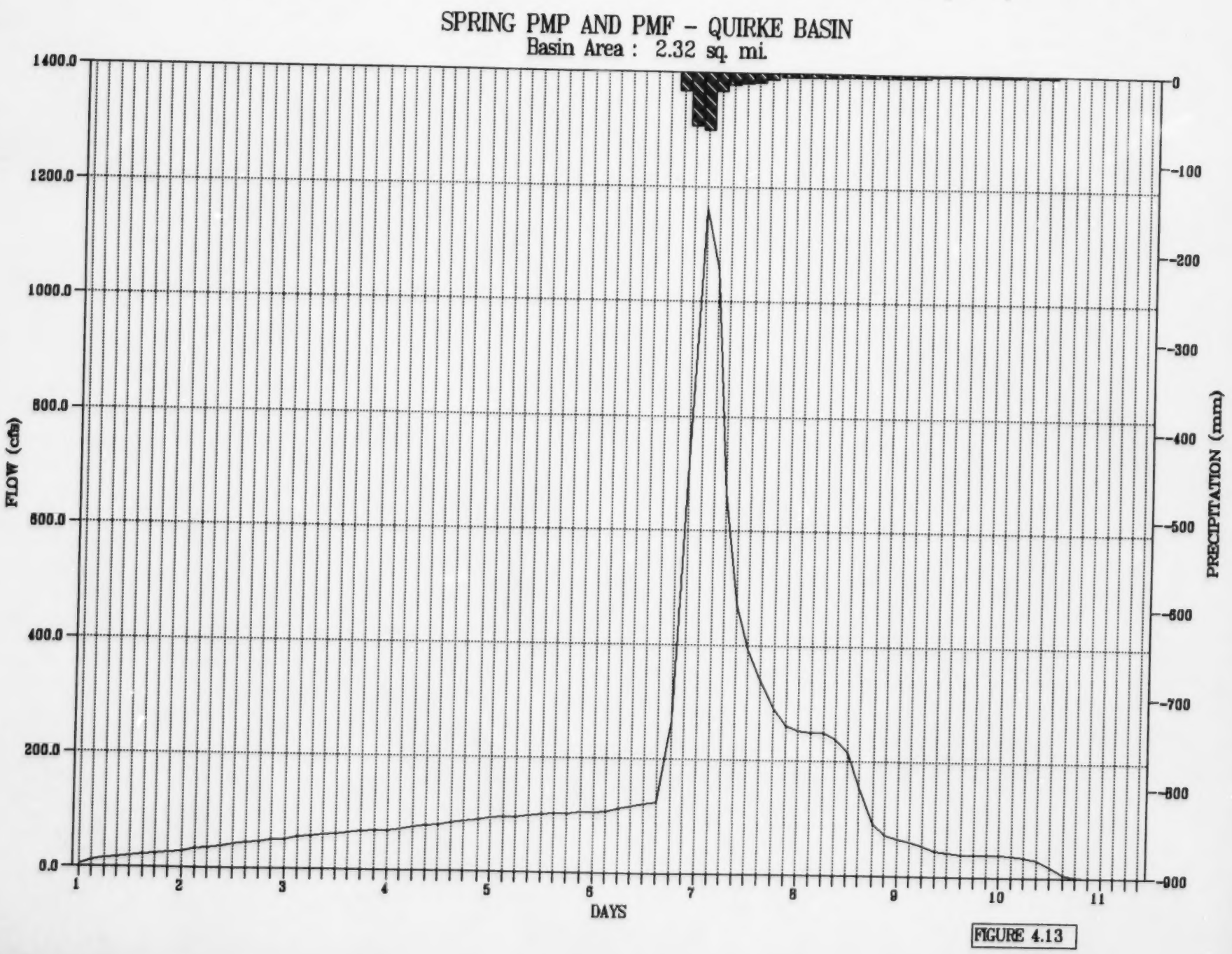

SPRING PMP AND PMF – QUIRKE BASIN
Basin Area : 2.32 sq. mi.
FLOW (cfs)
1400.0
1200.0
1000.0
800.0
600.0
400.0
200.0
0.0
PRECIPITATION (mm)
0
100
200
300
400
500
600
700
800
900
1
2
3
4
5
6
7
8
9
10
11
DAYS

FIGURE 4.13

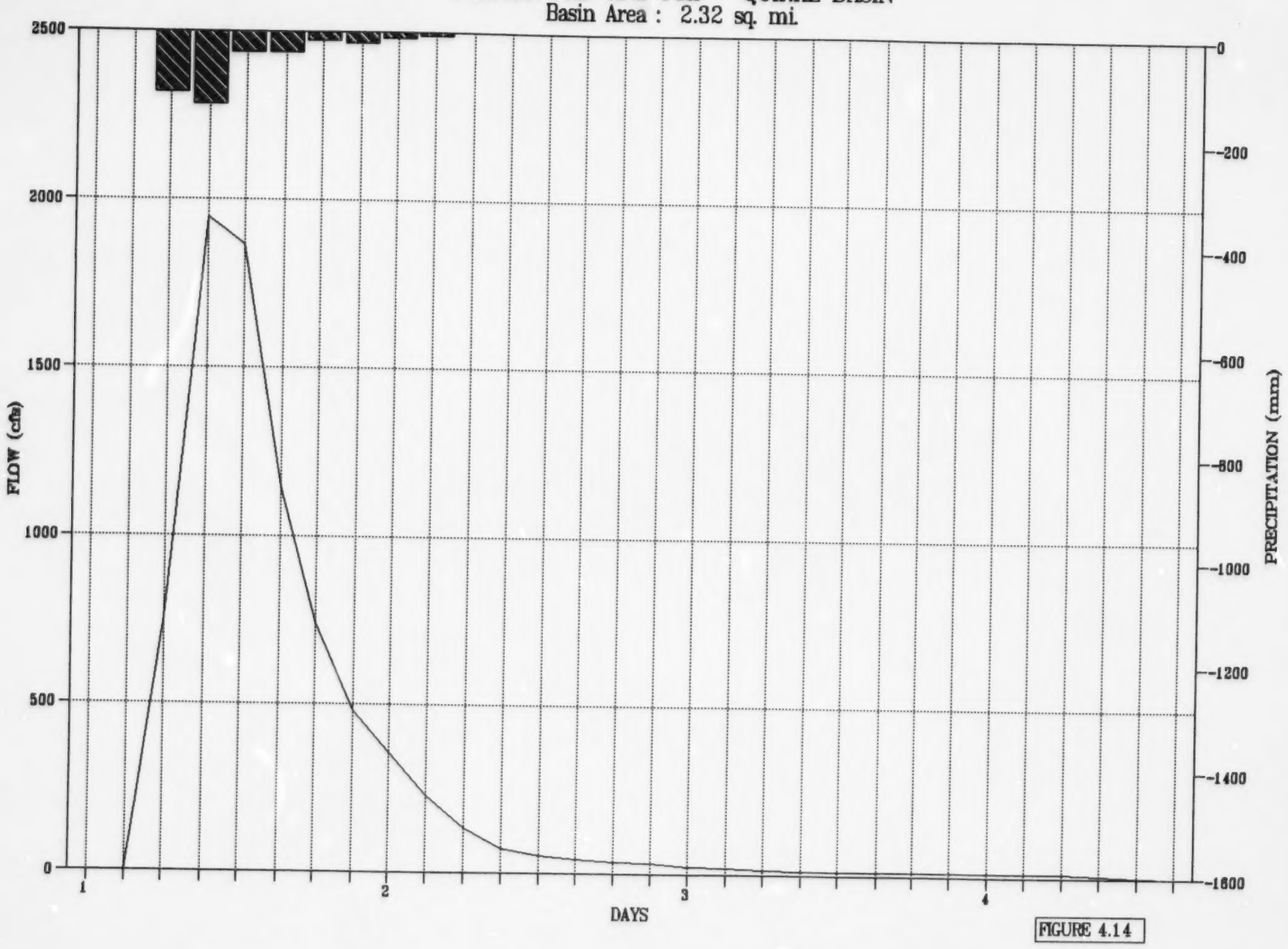

SUMMER PMP AND PMF - QUIRKE BASIN
Basin Area : 2.32 sq. mi.
FLOW (cfs)
2500
2000
1500
1000
500
0
PRECIPITATION (mm)
0
-200
-400
-600
-800
-1000
-1200
-1400
-1600
1
2
3
4
DAYS

FIGURE 4.14

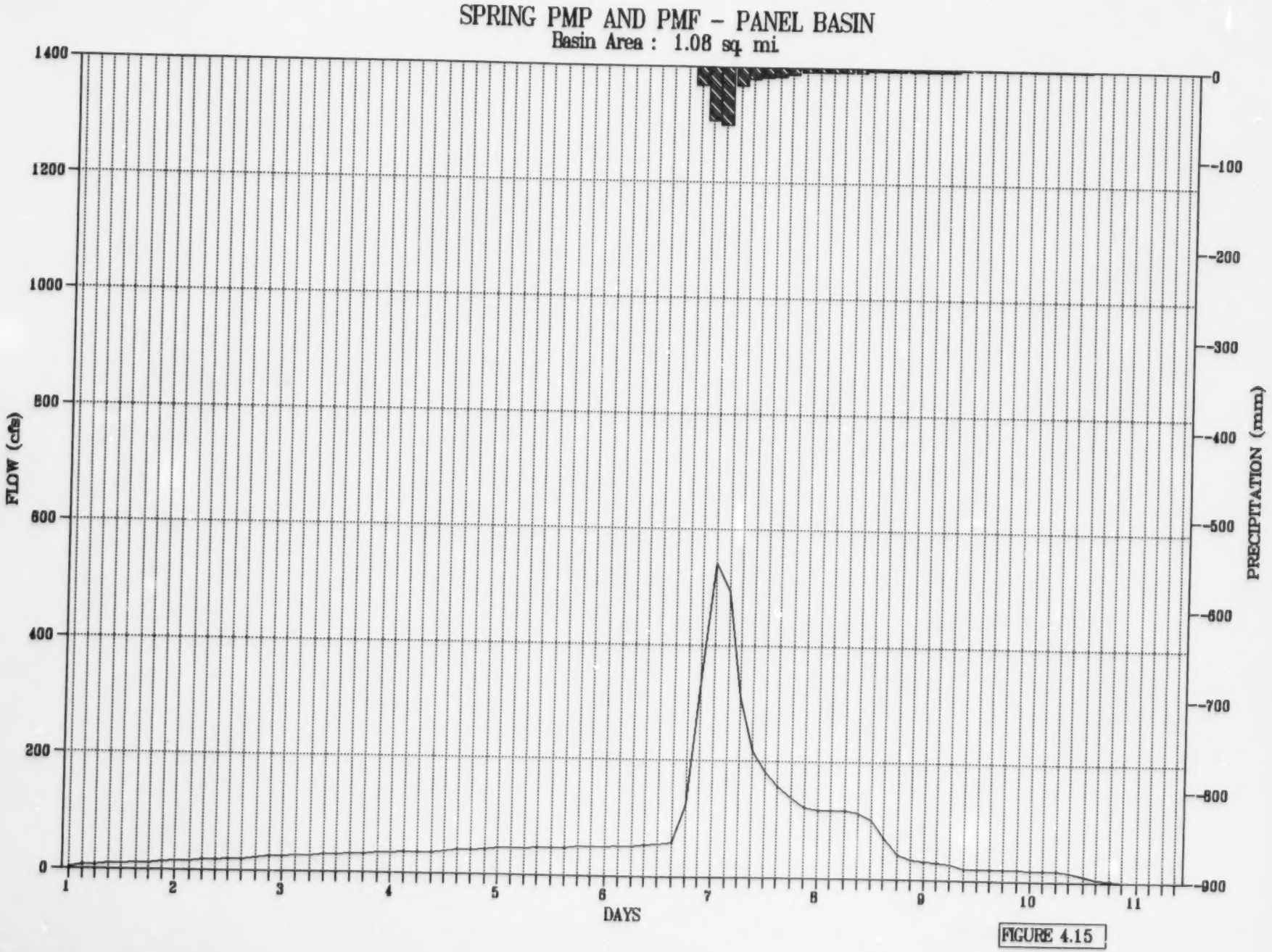

SPRING PMP AND PMF - PANEL BASIN
Basin Area : 1.08 sq. mi.
FLOW (cfs)
1400
1200
1000
800
600
400
200
0
PRECIPITATION (mm)
0
100
200
300
400
500
600
700
800
900
1
2
3
4
5
6
7
8
9
10
11
DAYS

FIGURE 4.15

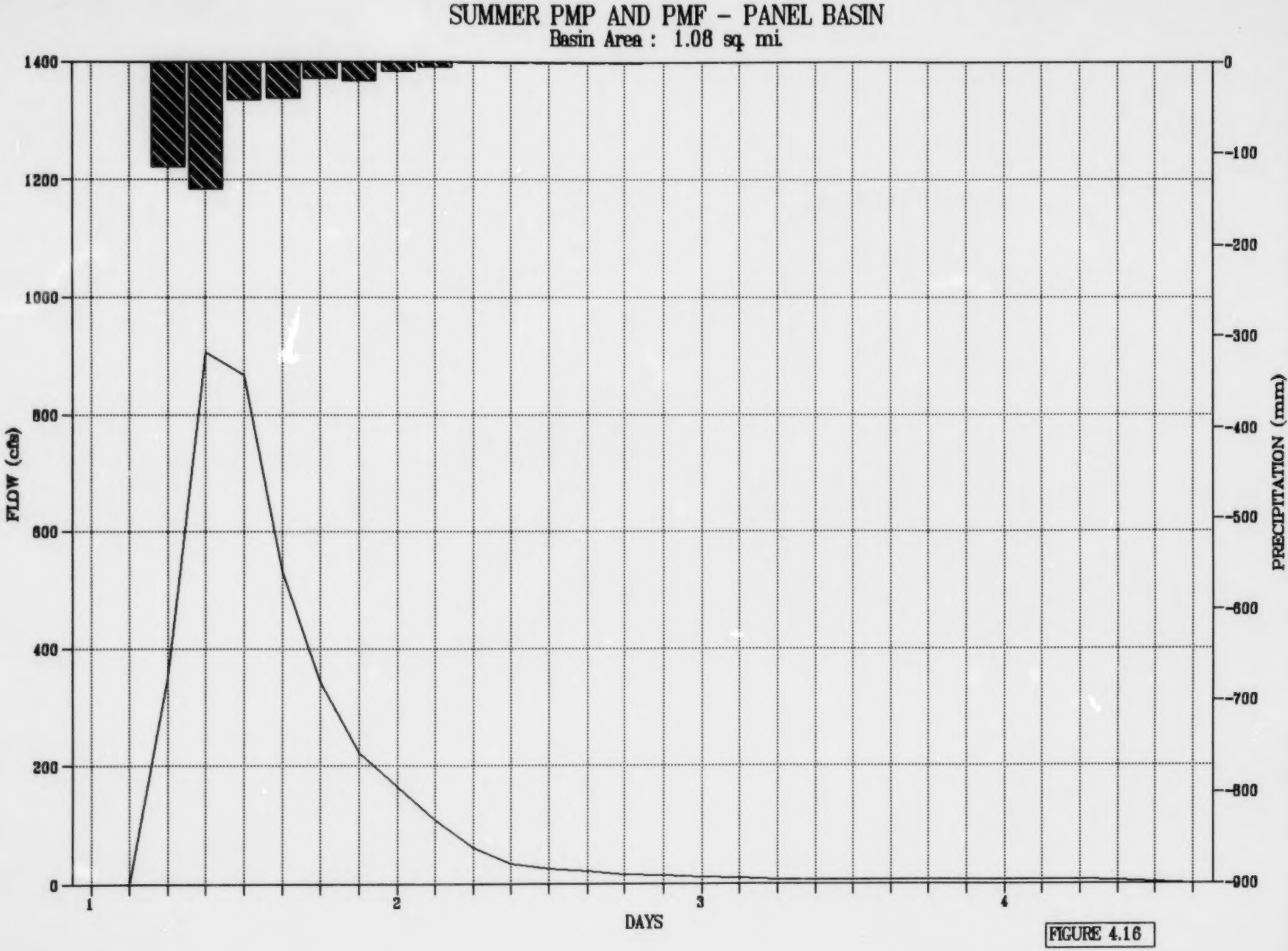

SUMMER PMP AND PMF - PANEL BASIN
Basin Area : 1.08 sq. mi.
FLOW (cfs)
PRECIPITATION (mm)
DAYS
1400
1200
1000
800
600
400
200
0
0
-100
-200
-300
-400
-500
-600
-700
-800
-900
1
2
3
4

FIGURE 4.16

If the estimated flood volume for the spring PMF at Red Rock Falls (local drainage area 832 sq. miles) as determined by the Dillon Study is used as a reference point, then the corresponding volume for a drainage area the size of the Quirke basin would be 1 639 ac-ft. This is less than the volumes produced in this study, see Table 4.8.

4.5 EVAPORATION

One of the more important factors which affect the maintenance of water in the lakes and cells is evaporation. Rio Algom Limited has kept records of pan evaporation for the years 1986-89 during the months of May to October. The recorded values are given in the table below.

TABLE 4.9

EVAPORATION RATES (mm/month)

Year	May	June	July	Aug.	Sept.	Oct.
1986	102	131	158	101	67	28
1987	161	198	91	139	61	67
1988	62	181	157	152	105	30
1989	141	126	230	151	93	34
Mean	116.5	159.0	159.0	135.7	81.5	39.7
Max	161	198	230	152	105	67

To use these values a pan coefficient must be applied, and the appropriate value corresponding to the latitude of the Quirke and Panel basins would be between 0.7 and 0.8. Also, in view of the short length of the evaporation data and the environmental rigorousness demanded of this study, it is recommended that the maximum monthly values should be adopted.

4.6 SEEPAGE LOSSES

It was initially intended to carry out the tailings area water balance using seepage losses established by Cumming Cockburn. However, based on examination of available reports, and review of the methodology used by Cumming Cockburn to determine seepage losses, it was considered more appropriate to use a somewhat modified approach to defining seepage losses which would also take into account the presently proposed configuration of water retention cells and basins and the variable water head across some of the containment dams and dykes. Sub-section 4.6.1 below provides a review and commentary on the seepage losses estimated by Cumming Cockburn. Sub-section 4.6.2 gives a description of the manner in which seepage losses were estimated or extrapolated by Shawinigan Lavalin.

4.6.1 Seepage Losses Estimated by Cumming Cockburn

Quirke Tailings Area

Extracts of the Cumming Cockburn hydrology reports provided by AECB were used to reproduce seepage losses estimated by Cumming Cockburn for the Quirke tailings area as summarized in Table 4.10 overleaf. The following comments are made regarding these seepage losses:

(i) Total external seepage losses of 132 IGPM were distributed by Cumming Cockburn to Gravel Pit Lake and Cells 1 to 5 based on proration of tailings/water surface plan area rather than by assigning seepage losses at individual external dams. This results in a non-representative distribution of external seepage loss. For

TABLE 4.10

APPROXIMATE REPRODUCTION OF SEEPAGE LOSSES ESTIMATED BY CUMMING COCKBURN FOR QUIRKE TAILINGS AREA

			External Seepage		Internal Seepage		
Location	External Dams	Normal Water Elevation (Feet)	Percent of Area (1) (%)	Seepage Loss (2) (IGPM)	Dam or Dyke	Length of Internal Dykes (3) (Feet)	Seepage Loss (4) (IGPM)
Gravel Pit Lake	Dam M	1315	17	23	Dam L		Not Reported
Cell 1	Dam K1 Dam K2	1310	26	34	Dyke 14A	3200	118
Cell 2	Dam J	1300	10	13	Dyke 15A	2800	103
Cell 3	Dam I Main Dam	1290	13	17	Dyke 16A	3000	110
Cell 4	None	1280	22	29	East Dyke	1500	55
Cell 5	Dam G1 Dam G2	1270	12	16			
		TOTAL	100	132			

(1), (2), (3) and (4) - see notes on Page 2 of 2

Notes:

1. Area percentages for tailings/water surfaces at Cells were estimated from Cumming Cockburn Figure ES-1 and are approximate only.

2. Total external seepage loss of 132 IGPM was distributed according to area percentages.

3. Lengths of internal dykes were obtained by scaling from Cumming Cockburn Figure ES-1 and are approximate only.

4. Approximate reproduction of Cumming cockburn internal seepage estimates excluding Dam L:

$q = ki\,A$
$q = 9.84 \times 10^{-3} \times 0.01 \times 6000$
$q = .59\ ft^3/min\ (3.68\ IGPM)$

Where:

q - seepage loss for 100 foot length of internal dyke on tailings
k - permeability of tailings
k = 5×10^{-3} cm/sec (9.84×10^{-3} ft/min)
i - hydraulic gradient
i = 0.01
A - area of tailings below dyke (assume average 60 feet depth)
A = 100 ft x 60 ft = 6000 ft^2

example, Cell 4 has a relatively high external seepage loss of 29 IGPM but does not involve an external dam (see Table 4.1).

(ii) Internal seepage losses through the tailings below the internal dykes were estimated by Cumming Cockburn based on a tailings coefficient of permeability of 5×10^{-3} cm/sec, a hydraulic gradient of 0.01 and the cross-sectional area of the tailings below the internal dyke (see note 4 on Table 4.10). Since the cross-sectional area of the tailings is not known, the seepage losses estimated by Cumming Cockburn were approximately reproduced by assuming that the average tailings depth below the internal dykes is 60 feet.

(iii) The internal seepage loss at Dam L between Gravel Pit Lake and Cell 1 was not reported in the extracts of the Cumming Cockburn report received. However these seepage losses are relatively minor since Dam L is a low permeability dam.

Panel Tailings Area

Seepage losses from the main basin of the Panel tailings area were reported by Cumming Cockburn to be 40 IGPM (3 l/sec). The extracts of the Cumming Cockburn report on the Panel tailings area did not include seepage losses from the south basin.

4.6.2 Seepage Losses Estimated by Shawinigan Lavalin

Quirke Tailings Area

Seepage losses estimated or extrapolated for the Quirke tailings area are summarized in Table 4.11 overleaf. These seepage losses are based on available information and in some cases extrapolation and judgement were used since detailed seepage analyses were not within the Shawinigan Lavalin scope of work. The following comments are made with respect to the seepage losses.

(i) The arrangement of cells and water levels in the cells used for estimating seepage losses are somewhat different from that used by Cumming Cockburn but are understood to represent current Rio Algom plans for cell construction.

(ii) External seepage losses are applied at the location of the appropriate dam in the direction of the hydraulic gradient produced by the differential head across the dam. Therefore external seepage losses are assumed to occur only at the lakes or cells which involve external dams.

(iii) Seepage losses for external dams (except for Dam M and the Main Dam) are based on estimates by Golder Associates applicable to a condition with the line of saturation at or below the tailings surface, i.e., no free water over the tailings surface or against the dams. No information has been provided on seepage estimates applicable to the condition of a flooded tailings surface. However the seepage

TABLE 4.11

SEEPAGE LOSSES ESTIMATED BY SHAWINIGAN LAVALIN
FOR QUIRKE TAILINGS AREA

Location	Normal Water Elevation (Feet)	External Seepage: External Dams	External Seepage: Seepage Loss (1) (IGPM)	Internal Seepage: Internal Dam or Dyke	Internal Seepage: Head Diff. (Feet)	Internal Seepage: Length of Internal Dyke (Feet)	Internal Seepage: Representative Seepage Loss (2) (IGPM)	Internal Seepage: Highest Likely Seepage Loss (2) (IGPM)
Gravel Pit Lake	1310-1315	Dam M	5-10(3)	Dam L	0-5		0 - 7 (4)	Not Applicable
Cell 14	1310	Dam K1 Dam K2	55 18	Dyke 14	13	3500	167	1670
Cell 15	1297	Dam J	14	Dyke 15	14	3000	155	1550
Cell 16	1283	Dam I Main Dam	7 30	Dyke 16 (East Dyke)	13	1500	72	720
Cell 17	1270	None		Dyke 17	2	1300	10	100
Cell 18	1268	Dam G1 Dam G2	20 11					
		TOTAL	160-165					

(1), (2), (3) and (4) - see notes on Page 2 of 2

Notes:

1. Seepage loss is from Golder Associates Report 871-1112, March, 1989 (Table B-3) except for Dam M (see Note 3) and Main Dam. Seepage of 30 IGPM at Main Dam has been extrapolated from other dams.

2. Seepage loss for Internal Dykes on tailings estimated as follows:

   $q = 0.6\ kHL$
   $q = 0.6 \times 9.84 \times 10^{-4} \times 13 \times 100$
   $q = 0.768\ ft^3/min$ (4.78 IGPM)

H	q (for L = 100 ft)	
	Representative Value	Highest Likely Value
2 ft.	0.74 IGPM	7.4 IGPM
13 ft.	4.78 IGPM	47.8 IGPM
14 ft.	5.15 IGPM	51.5 IGPM

   Where:

   q - seepage loss below internal dyke
   0.6- shape factor for seepage flow through tailings under low permeability internal dyke without cutoff.
   k - permeability of tailings
   k = 5×10^{-4} cm/sec (9.84×10^{-4} ft/min) - representative value or
   k = 5×10^{-3} cm/sec (9.84×10^{-3} ft/min) - highest likely value
   H - head difference across internal dyke
   H = 2, 13 or 14 ft.
   L - length of internal dyke
   L = 100 ft

3. External seepage at Dam M (extrapolated from other dams) is 5 IGPM for Gravel Pit Lake at El. 1310 and 10 IGPM for Gravel Pit Lake at El. 1315.

4. Internal seepage at Dam L is 7 IGPM for assumed 5 feet head difference (Golder Associates Report 871-1112, March 1989, Table B-3) and is 0 for no head difference.

estimates are considered by Golder Associates to be conservatively high since they were based on ungrouted bedrock. Therefore the available seepage estimates by Golder Associates are considered to be sufficiently representative and were used for the water balance study.

(iv) External seepage loss at Dam M (outlet of Gravel Pit Lake) has been roughly established by extrapolation of seepage estimates from other dams. This extrapolation has taken into consideration that Dam M has a glacial till core and partial cutoff (cutoff does not extend to bedrock in the central part of the dam) and that the bedrock below Dam M has not been cement grouted. Also, as noted on Table 4.11, the seepage at Dam M would vary with the water level elevation in Gravel Pit Lake.

(v) External seepage at the Main Dam has been roughly established as 30 IGPM by extrapolation of seepage estimates from other dams. Seepage loss estimates of 1 IGPM through the embankment and 3 IGPM through the slurry wall are given in Golder Associates Report 901-1802 dated March, 1990 and titled "Geotechnical Investigation and Design Recommendations, Main Dam Remediation, Quirke Mine Tailings Impoundment". The above report does not contain an estimate of seepage loss through the bedrock below the Main Dam and the amount of 30 IGPM includes an allowance for this purpose.

(vi) Seepage losses for internal dykes on tailings have been estimated by Shawinigan Lavalin as outlined in note 2 of Table 4.11. Representative seepage losses are based on a tailings coefficient of permeability of 5×10^{-4} cm/sec which is judged to be an average value. However the coefficient of permeability could be significantly higher and could be as high as the value of 5×10^{-3} cm/sec used by Cumming Cockburn. This higher value of tailings permeability results in seepage losses at internal dykes which are ten times higher than the representative seepage losses. Therefore the water balance was carried out for the following two values of tailings permeability:

- Representative Value of Tailings Permeability = 5×10^{-4} cm/sec

- Highest Likely Value of Tailings Permeability = 5×10^{-3} cm/sec.

Lower values of tailings permeability (as low as about 5×10^{-5} cm/sec) could occur particularly in areas where the fine fraction of the tailings have been preferentially deposited. However the lowest likely permeability range of the tailings does not have to be considered in the water balance study since this is not a critical condition for low drought flow or extreme flood flow scenarios. The trial flooding of Cell 14 and associated monitoring proposed by Rio Algom for the summer of 1990 will of course provide valuable information on actual seepage losses at internal dykes.

(vii) Figure 4.17 shows a plot of coefficient of permeability of tailings versus depth below tailings surface for the Quirke tailings area based on data contained in Golder Associates Report 871-1112 dated March, 1989. Also shown on this figure are the three tailings permeability classifications identified by Shawinigan Lavalin, i.e., lowest likely value (5×10^{-5} cm/sec), representative value (5×10^{-4} cm/sec) and highest likely value (5×10^{-3} cm/sec). It should be noted, with reference to Tables 4.10 and 4.11, that internal dyke seepage losses of about 4 IGPM per 100 feet length were estimated by Cumming Cockburn for a tailings permeability of 5×10^{-3} cm/sec and that corresponding seepage losses of about 50 IGPM per 100 feet length of dyke were estimated by Shawinigan Lavalin for the same tailings permeability of 5×10^{-3} cm/sec.

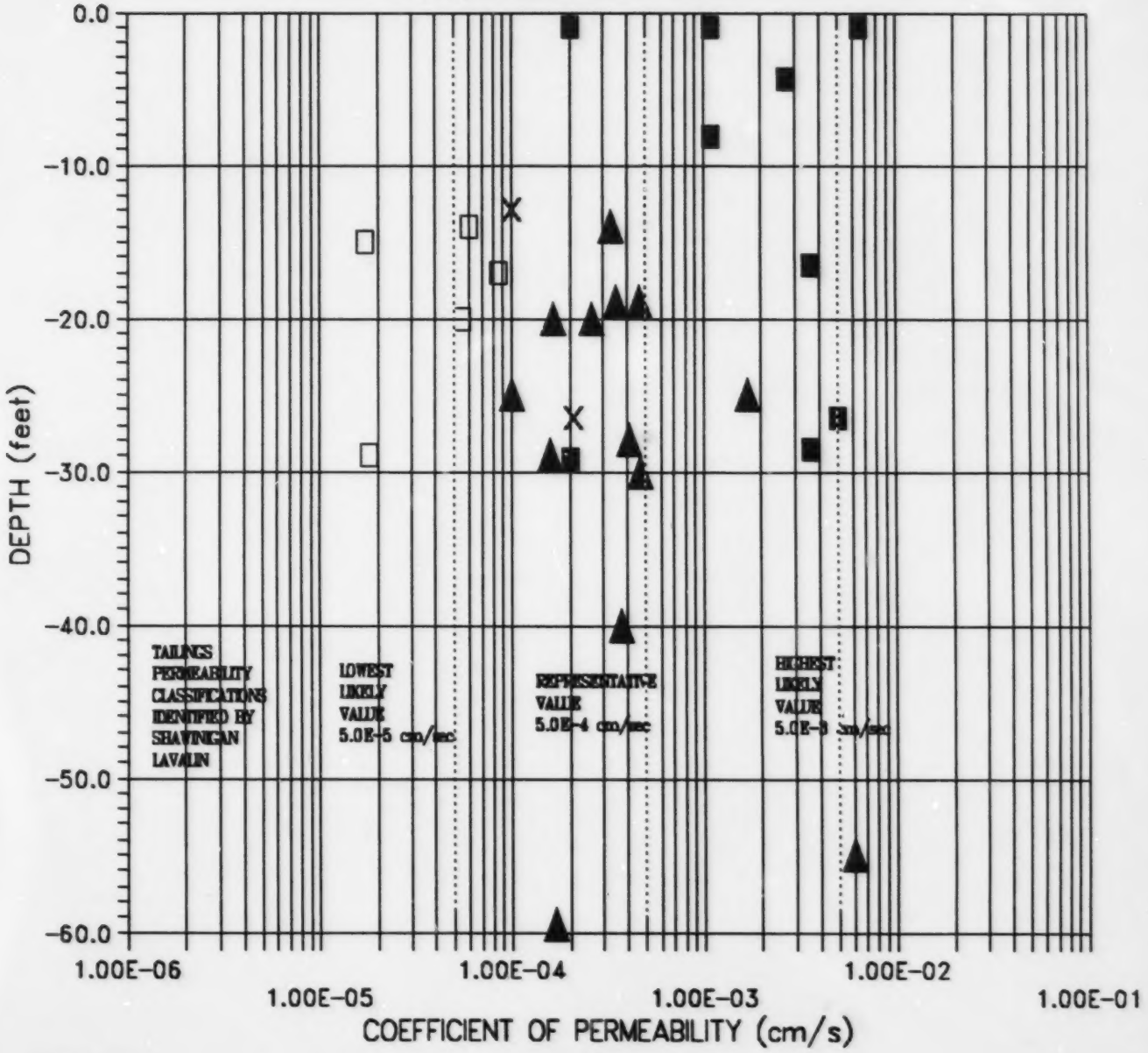
It is therefore considered that the average hydraulic gradient of 0.01 used by Cumming Cockburn is too low for the seepage conditions involved at the internal dykes.

(viii) The internal seepage loss at Dam L from Gravel Pit Lake to Cell 14 will vary with the head difference across the dam as noted in Table 4.11. The seepage estimate of 7 IGPM given by Golder Associates applies for seepage flow from Cell 14 to Gravel Pit Lake, i.e., for water level in Cell 14 higher than in Gravel Pit Lake (head difference not known but assumed to be about 5 feet). The same value of seepage is assumed to apply with a reverse gradient, i.e., water

FIGURE 4.17

COEFFICIENT OF PERMEABILITY OF TAILINGS VS. DEPTH BELOW TAILINGS SURFACE

FOR QUIRKE TAILINGS AREA



LEGEND

- ▲ PIE 2 RESPONSE TEST - SAND TAILINGS
- □ PIE 2 RESPONSE TEST - SILT TAILINGS
- ■ GRADATION CORRELATION - SAND TAILINGS
- × GRADATION CORRELATION - SILT TAILINGS

NOTES

1. DATA FROM GOLDER REPORT 871-1112, MARCH, 1989
2. GRADATION CORRELATION BASED ON $K=D_{10}^{2}$

   K = COEFF. OF PERM -cm/sec

   D_{10} = GRAIN DIAMETER AT 10 % PASSING -mm

level in Gravel Pit Lake is 5 feet higher than in Cell 14.

Panel Tailings Area

The Panel tailings area water balance was carried out using a seepage loss of 40 IGPM from the main basin and this value of seepage loss was obtained from the Cumming Cockburn hydrology report. About half of this amount (20 IGPM) was assumed to report to the south basin by seepage primarily at Dam D and proposed Dam H. The other half of this amount (20 IGPM) was assumed to occur mainly at Dams E and B on the northeast side of the main basin and therefore would bypass the South Basin. Seepage losses from the South Basin itself were assumed by Shawinigan Lavalin to be 20 IGPM and to occur mainly at Dams A and F.

5. SIMULATION STUDIES

5.1 FLOOD FLOWS

In order to determine maximum levels in each component of the two basins, and thus to verify the adequacy of the weirs in internal dykes and spillways, estimated probable maximum floods (PMF) for each tailing basin were routed through their systems.

As the estimated floods for spring and summer conditions have different characteristics, in terms of hydrograph shape, peak and base flow, the simulations were done for both spring and summer conditions.

5.1.1 Quirke Basin

Maximum probable floods for the Quirke tailings basin, at Gravel Pit Lake have been derived for spring and summer conditions (Section 4). The spring flood includes a significant snow melt component, however the precipitation component is less than for the summer flood. Estimated peak inflows are:

	3 Hour Maximum Cfs
Spring Flood	1163
Summer Flood	1947

These base flows are calculated for the 2.32 square mile drainage area at Gravel Pit Lake. Runoff inflows to each cell are based on the ratio of cell drainage area to the Gravel Pit Lake drainage area.

Each of these flood hydrographs have been routed through the Quirke 5-cell tailings basin to determine the maximum levels in each cell, and to demonstrate which event is more critical. The results are summarized as follows:

	Maximum Water Level - Ft.				
Cell	Dyke/Dam Crest Level	Spillway/ Weir Crest Level	Spring Flood	Summer Flood	(Summer) Minimum Freeboard
14	1314	1311.5	1312.13	1312.01	1.99
15	1301	1298.5	1299.27	1299.88	1.12
16	1287	1284.5	1285.65	1285.60	1.40
17	1274	1271.5	1272.71	1272.63	1.37
18	1273	1268.5	1269.37	1269.25	3.75

These analyses assume no flow from Gravel Pit Lake into Cell 14. Weir discharges, evaporation and other losses are as described in Section 3.

The summer and spring flood events provide similar values, however significant freeboard remains when peak levels occur in each cell.

The significantly higher spring flood volume, due to the snow melt component offset the higher peak of the summer PMF.

Initial levels correspond to a 2 foot depth over the tailings surface, i.e., to the maximum controlled water level proposed by Rio Algom.

It must be noted that the long term simulations for the low flow analyses showed that levels would frequently be above these values unless a supplementary level control facility was added at each dyke.

5.1.2 Panel Basin

As is the case of Quirke Lake, separate estimates have been made for probable maximum floods based on occurrence in the spring, with a snow melt component, and in the summer.

The estimated peak inflows are:

	3-Hour Maximum Cfs
Spring Flood	541
Summer Flood	906

These are for the combined Strike Lake and South Basin drainage areas, of 0.99 square miles.

These floods were routed through the two cell Panel Tailings Basin and the following peak levels were estimated:

	Maximum Water Level - Ft.				
Cell	Dyke/Dam Crest	Spillway/ Weir Crest	Spring Flood	Summer Flood	(Summer) Minimum Freeboard
Strike	3365	3360	3362.07	3362.31	2.69
South Basin	3322	3316/3319	3318.51	3318.43	3.57

These analyses assume no inflow from Frayn Lake into Strike Lake, and weir discharges, evaporation and other losses are as described in Section 3.

The results show that the spring and summer floods, despite their different characteristics, would yield similar peak levels in each basin, and that significant freeboard could remain during either PMF event.

As at the Quirke facility, initial water levels were assumed as the maximum controlled water level, i.e., providing a 2 foot depth over the tailings surface.

The long term simulations showed that actual starting levels could be marginally above the assumed level, (by less than 0.4 feet), however this would only reduce the above remaining freeboard levels by less than 0.3 feet.

5.2 LOW FLOW CONDITIONS

The objective of the low flow simulations is to estimate conditions that could occur under average or typical low flows, and under extreme drought conditions. These tests show under what conditions the tailings surfaces could be exposed to oxidation when water levels in individual cells fall below the tailings surface.

The available historic flow record for each site obtained from gauges 2CD2 and 2CD6, and adjusted for drainage area, extends from January 1978 to December 1988, i.e., for 132 months. During the sequence the driest short term period occurs during 1987. This applies to low flow period averages up to 150 days.

The simulations were carried out in two parts:

- Using the historic record adjusted to the site;

- Using a 120 day period for extreme events.

No upstream makeup water supply was included.

No level control system, to maintain depths at not more than 2 feet over the tailings surface, was assumed.

Recorded monthly flows for the period 1978-88 and 1987 daily flows are shown in Tables 5.1 and 5.2 respectively. These are for the drainage area between 2CD2 and 2CD6, and are the base flows used for this study.

Table 5.1

STATION 02CD006 - STATION 02CD002

Mean Monthly Flows (cms)

	JAN	FEB	MAR	APR	MAY	JUNE	JULY	AUG	SEPT	OCT	NOV	DEC
1967										0.95	1.85	1.20
1968				3.23	1.37	1.34	0.77	0.75	2.26	1.71	1.29	
1969				2.88	1.73	1.54	0.99	0.82				1.11
1970			1.23	2.26	1.81	2.90	0.63	0.74	1.53	1.87	2.23	
1971						1.11	0.44					1.14
1972					2.66	0.65	0.52				1.01	0.85
1973	0.89	0.79	1.81	2.49	1.75	0.89	0.77	1.50	0.88	0.95	0.77	0.82
1974	0.70	0.55	0.78	3.14	1.68	1.00	0.60	0.41	0.54	0.88	1.45	0.96
1975						1.83		0.49	0.63			
1976						0.80	0.48					
1977									0.74	1.68	2.52	1.35
1978	0.81	0.72	0.87	1.99	2.34	0.96	0.67	0.64	1.32	1.88	1.19	0.87
1979	0.54	0.56	1.90	3.07	1.91	1.00	0.88	0.79	0.55	1.78	2.10	1.28
1980	1.49	0.89	0.51	2.40	1.48	0.79	0.68	0.63	0.79	1.12	1.02	0.67
1981	0.55	1.42	1.51	2.95	1.03	1.23	0.85	0.37	0.50	0.75	0.69	0.81
1982	0.83	0.59	0.86	2.99	2.09	0.75	0.67	0.46	1.13	2.34	2.12	2.03
1983	1.53	1.18	1.66	2.78	2.11	1.69	0.50	0.56	0.65	1.34	0.45	0.28
1984	0.22	0.42	0.81	2.31	1.21	0.72	0.47	0.63	1.25	1.21	2.33	2.04
1985	1.14	0.65	1.19	4.28	2.08	0.82	0.75	0.77	0.80	1.23	1.53	0.73
1986	0.50	0.41	1.17	2.76	1.07	0.82	0.32	0.76	0.87	1.57	0.99	0.82
1987	0.76	0.56	1.63	2.64	1.04	0.75	0.40	0.31	0.40	0.69	0.88	1.26
1988	0.82	0.72	0.89	5.15	1.51	0.71	0.48	0.87	0.97	1.97	4.22	1.42
AVG	0.83	0.73	1.20	2.96	1.70	1.12	0.62	0.68	0.93	1.41	1.59	1.09

Table 5.2

STATION 02CD006 - STATION 02CD002

Daily Flows: 1987 (cms)

	JAN	FEB	MAR	APR	MAY	JUNE	JULY	AUG	SEPT	OCT	NOV	DEC
1	0.86	0.60	0.72	4.25	0.96	1.14	0.59	0.27	0.39	0.61	0.79	1.37
2	0.89	0.59	0.71	4.10	0.94	1.04	0.51	0.26	0.41	0.89	0.75	1.39
3	0.80	0.57	0.70	4.13	0.83	0.96	0.44	0.27	0.40	0.93	0.84	1.30
4	0.77	0.56	0.66	4.14	0.83	0.94	0.41	0.26	0.39	0.91	0.98	1.18
5	0.78	0.57	0.34	4.15	0.82	0.87	0.41	0.28	0.38	0.85	1.01	1.10
6	0.72	0.56	0.30	4.15	0.77	0.84	0.45	0.31	0.35	0.80	0.99	1.06
7	0.75	0.56	0.57	4.31	0.77	0.83	0.41	0.32	0.35	0.80	0.94	1.03
8	0.76	0.53	0.75	4.35	0.74	0.84	0.41	0.31	0.34	0.69	0.88	1.04
9	0.76	0.55	0.76	4.38	0.68	0.86	0.48	0.29	0.38	0.59	0.83	1.16
10	0.81	0.55	0.79	4.42	0.60	0.81	0.48	0.30	0.37	0.51	0.78	1.55
11	0.90	0.56	0.82	4.43	0.56	0.72	0.41	0.28	0.37	0.48	0.74	2.07
12	0.88	0.56	0.83	4.50	0.65	0.85	0.46	0.29	0.42	0.46	0.72	1.92
13	0.89	0.33	0.84	4.32	0.87	0.93	0.45	0.36	0.43	0.43	0.67	1.79
14	0.70	0.08	0.88	3.92	0.80	0.82	0.50	0.33	0.42	0.42	0.67	1.68
15	0.60	0.44	0.89	1.96	0.88	0.70	0.46	0.33	0.41	0.42	0.66	1.53
16	0.78	0.50	0.91	1.80	0.87	0.65	0.41	0.33	0.37	0.42	0.66	1.49
17	0.83	0.47	0.96	1.46	0.99	0.62	0.35	0.32	0.30	0.51	0.75	1.45
18	0.81	0.46	0.98	1.31	1.19	0.61	0.34	0.31	0.29	0.62	0.96	1.32
19	0.83	0.48	1.15	1.30	1.22	0.55	0.33	0.35	0.30	0.65	1.02	1.23
20	0.83	0.57	1.61	1.20	1.09	0.60	0.34	0.34	0.37	0.66	1.03	1.21
21	0.81	0.66	1.96	1.15	1.04	0.57	0.39	0.35	0.36	0.68	1.03	1.18
22	0.69	0.68	2.02	1.16	1.27	0.61	0.44	0.34	0.37	0.68	0.97	1.18
23	0.69	0.70	2.30	1.08	1.81	0.69	0.39	0.33	0.42	0.69	0.92	1.14
24	0.70	0.71	2.69	1.07	1.81	0.56	0.32	0.30	0.44	0.72	0.94	1.13
25	0.66	0.72	2.84	1.02	1.53	0.58	0.37	0.29	0.49	0.77	0.94	1.12
26	0.69	0.73	3.09	1.03	1.31	0.61	0.35	0.28	0.53	0.79	0.91	1.11
27	0.72	0.74	3.58	1.03	1.34	0.62	0.33	0.28	0.51	0.83	0.91	1.01
28	0.67	0.74	3.76	1.06	1.39	0.70	0.30	0.28	0.49	0.85	0.91	0.96
29	0.67		3.80	1.10	1.28	0.68	0.28	0.27	0.52	0.89	0.94	0.88
30	0.66		4.12	1.01	1.24	0.65	0.29	0.33	0.53	0.88	1.22	0.82
31	0.62		4.20		1.15		0.28	0.38		0.86		0.75
AVG	0.76	0.56	1.63	2.64	1.04	0.75	0.40	0.31	0.40	0.69	0.88	1.26

Long Term Sequence

The purpose of the simulation with the historic record is to demonstrate conditions that could occur under normal low flow conditions. No assessment is made of whether this sequence (1978-88) represents average long term conditions or whether these flows were above or below the long term average.

The long term simulation for both basins was done using monthly data. It shows that for the Quirke basin levels do not necessarily rise to the normal maximum in each year, whereas levels in the Panel basin do reach the normal maximum each year.

The long term simulation also provides a check on the initial conditions (water levels in each cell) to be used for testing more severe seasonal drought events.

120 Day Sequence

The simulations using a shorter term dry period were based on a 120 continuous calendar day historical sequence from June 14, 1987, to provide the distribution of flows over the period. It may be noted that 1987 is the driest of the 11 years of continuous record, and this year has a calculated return period of 18 years.

Tests using this sequence were made with initial cell water levels taken from the long term simulation, and by adjusting the volume of runoff over the 120 days to also simulate a 100 year return period event.

Examination of average monthly flows showed that 1987 contained the lowest monthly flows during the period May to October. Minimum periods and average flows are:

Days	Period	Flow (cfs) For 2CD6-2CD2
60	July 24 - September 21	11.68
90	July 2 - September 29	12.88
120	June 20 - October 17	15.20
150	June 14 - November 11	17.60

The 120 day sequence of actual flows in 1987 from June 20-October 17 was used to give the distribution of flows over the period.

The 120 day sequence for the 100 year condition was taken by applying the ratio of average flows over the 120 day period for the 100 year return period and the 18 year (1987) period to the actual 1987 distribution.

Comparison of these values (Figure 4.6) showed the ratio of 120 day runoff for a 100 year return period was 80.4% of the 1987 actual flows.

Evaporation

Evaporation data obtained by Rio Algom are discussed in Section 4.5. For the water balance simulations it is assumed that estimated runoff values already take into account evapotranspiration losses. Consequently for the cell water surfaces it is only necessary to add the difference between the total evaporation and the evapotranspiration already accounted for.

As the critical dry period is in 1987, monthly evapotranspiration rates were estimated, using temperature data for Elliot Lake (AES Station 6052268), by the Thornthwaite method.

Estimated evapotranspiration rates are:

Month	Mean Temperature °C	Evapotranspiration mm/Month
May	11.5	45.0
June	17.5	85.5
July	20.5	110.3
August	17.8	82.3
September	13.8	48.2
October	5.4	10.5

The estimated incremental evaporation over the water surfaces was then estimated by applying a pan coefficient of 0.8 to the Rio Algom data, and subtracting the estimated evapotranspiration.

Month	Rio Algom Measured Evaporation mm	x 0.80 mm	Evapotranspiration mm	Incremental Evaporation Over Water Surface mm/Mo	Ft/day
May	161	129	45	84	0.0091
June	198	158	85	73	0.0079
July	230	184	110	74	0.0080
August	152	122	82	40	0.0042
September	105	84	48	36	0.0039
October	69	54	11	43	0.0047

(The factor of 0.8 is appropriate for shallow lakes).

No assessment has been made on whether growth of vegetation in the tailings cells would affect the estimates of incremental evaporation over the cell water surfaces.

It may be noted that the evaporation over water is estimated as approximately 90% more than evapotranspiration. Consequently the evapotranspiration from the newly created wet lands in the tailings cells would have to increase by this amount before increasing the evaporation losses above those presently assumed.

5.2.1 Quirke Basin

Long Term

The simulation of conditions in the 5 cell Quirke tailings basin for the period 1978 to 1988 yielded the following maximum and minimum levels.

Water Levels - Ft.

Cell	Maximum Level	Minimum Level	Tailings Level
14	1311.02	1308.94	1308
15	1298.21	1297.21	1295
16	1284.52	1283.62	1281
17	1271.52	1270.74	1268
18	1268.54	1268.51	1266

The analyses are based on the representative or medium estimated seepage from the external dams.

Short Term

The simulation of the 120 day sequences provided the following minimum levels:

Cell	Start Level(1)	Minimum Levels 1987(2)	Minimum Levels 100 Yr. Even	Tailings Surface
14	1309.90(2)	1309.09	1308.71(4)	1308
15	1297.45(2)	1297.08	1296.84(4)	1295
16	1283.90(2)	1283.61	1283.53	1281
17	1271.02(1)	1270.74	1270.67	1268
18	1268.51	1268.51	1268.50	1266

(1) Start level corresponds to level for June 20, 1987, from long term simulation.

(2) It may be noted that these are above the normal maximum due to preceding conditions.

(3) End of dry period is October 17, 1987, on long term sequence.

(4) Indeterminate answer - minimum may be after 120 day sequence.

Long and short term simulations ignore effect of possible installation of supplementary weirs to maintain normal maximum depth at 2 feet over the tailings surface.

It should be noted that controlling maximum levels to a lower level would correspondingly reduce minimum levels.

5.2.2 Panel Basin

Long Term

The simulation of conditions in the 2 cell Panel tailings basin for the period 1978-88 showed that water levels would have returned to normal maximum level every year, and that the tailings surface would always have remained flooded.

Water Levels - Ft.

Cell	Maximum Level	Minimum Level	Tailings Surface
Strike Lake	3360.30	3359.52	3358
South Basin	3316.40	3315.95	3314

Short Term

The simulation of the 120 day sequence provided the following minimum water levels:

Cell	Start Level	Minimum Levels		Tailings Surface
		1987	100 Yr. Event	
Strike Lake	3359.89	3359.46	3359.32	3358
South Basin	3316.01	3315.99	3315.97	3314

5.3 SEEPAGE ESTIMATES

The long term simulation of low runoff effects for both basins was based on the median or most probable seepage rate (5×10^{-4} cm/sec), for internal and external dams.

At the Quirke mine facility, seepage under the intermediate dykes is a major factor in the water balance for each cell. This is particularly true for cells 14 and 15 which have the longest dykes.

The analyses for the Quirke mine tailings impoundments showed that adequate storage was provided by maintaining the weir inverts at 3.5 feet (including a 1.5 foot thick permeable base) above the top of the tailings.

As the actual seepage rates are unknown, it was considered useful to determine by trial and error the seepage rate that would result in the storage over the tailings being fully depleted, i.e., the maximum seepage rate that could be tolerated.

This was determined as less than 50% more than the representative value described in Section 4.6.

6. SUMMARY AND CONCLUSIONS

6.1 GENERAL CONSIDERATIONS

The objectives of this study are to -

- Assess the risk of overtopping internal or external dykes during an extreme flood event;

- Determine whether the natural runoff available to the tailings areas will be sufficient to maintain them in a saturated state during extreme drought conditions,

for the Rio Algom Quirke and Panel mine tailings areas, based on proposed arrangements for close out of these facilities.

The scope of work carried out to meet these requirements has included:

- Development of an estimated probable maximum flood for each basin, using extreme precipitation data, climatic and physiographic data, and rainfall-runoff modelling;

- Development of a flood frequency distribution for each basin, using representative recorded runoff values for the region;

- Examination of representative recorded flows to determine minimum flow regimes and extrapolation of these to extreme values;

- Development of a flow routing model for the simulation of flows and levels in each tailings basin, and sub-basin, for extreme flood and low flow conditions;

- Simulation of flows and levels for a range of flood and drought conditions.

6.2 QUIRKE TAILINGS BASIN

Flood Conditions

Estimates have been made for probable maximum floods occurring in the spring, with a snow melt component, and the summer. In addition, to provide partial verification of the estimated PMF, a frequency analysis has been made of flood peaks. All these estimates are referenced to runoff into Gravel Pit Lake (drainage area 2.32 square miles). Estimated peak flows are:

PMF - summer (3 hour average) 1947

PMF - spring (3 hour average) 1163

Flow routing studies for the two alternative PMF events shows that levels through the tailings complex would be similar, and that resulting freeboards on the internal dykes would always exceed 1 foot. Minimum freeboard would occur at dyke 15.

Cell	Maximum Water Level - Ft. Dyke/Dam Crest Level	Spillway/ Weir Crest Level	Spring Flood	Summer Flood	(Summer) Minimum Freeboard
14	1314	1311.5	1312.13	1312.01	1.99
15	1301	1298.5	1299.27	1299.88	1.12
16	1287	1284.5	1285.65	1285.60	1.40
17	1274	1271.5	1272.71	1272.63	1.37
18	1273	1268.5 (1)	1269.37	1269.35	4.75

Flood routing assumes that the relatively wide trapezoidal weirs are provided at each internal dyke.

Initial levels are assumed as corresponding to the controlled maximum depth of 2 feet over the tailings surface. This in turn requires supplementary weirs for level control.

Low Flows

Under minimum historic flow conditions - approximately estimated to have a return period of 18 years, the water levels in all cells would have remained above the tailings surfaces.

However this is contingent on:

- Minimal seepage losses at the external dykes;
- Median seepage under the weirs in the internal dykes;
- Estimated evaporation values;

- No level control system, i.e., depths are allowed to increase above the 2 foot nominal maximum depth.

It is relevant to note that some preliminary results using higher evaporation values showed some exposure of the tailings surfaces in Cell 14, although other cells would remain flooded.

Similarly adding only 50% to the seepage rates at the internal dams would also result in estimated water levels below the tailings surfaces.

6.3 PANEL TAILINGS BASIN

Flood Conditions

As for the Quirke basin, estimates have been made of the probable maximum flood for spring and summer conditions, and these floods have been routed through the Panel tailings complex. In addition a flood frequency analysis has been carried to define the 10 000 year return period flood. The flood estimates are as follows:

PMF - summer (3 hour average) 906 cfs

PMF - spring (3 hour average) 541 cfs

These are for the Strike Lake and South basin drainage area, (0.99 sq. miles).

As for the Quirke basin, flow routing studies for the two alternative PMF events shows that levels through the tailings complex would be similar, and that resulting freeboards on the dykes would always exceed 2.7 feet. Minimum freeboard would occur at Strike Lake.

	Maximum Water Level - Ft.				
Cell	Dyke/Dam Crest	Spillway/ Weir Crest	Spring Flood	Summer Flood	(Summer) Minimum Freeboard
Strike	3365	3360	3362.07	3362.31	2.69
South Basin	3322	3316/3319	3318.51	3318.43	3.57

The estimated maximum levels in Strike Lake assume the spillway channel into the South Basin is at elevation 3360 feet and has a width of 10 feet.

Low Flows

The simulation of water levels in the Panel 2 - cell complex based on the estimated historic record for 1978 to 1988 shows that water levels would have returned to the normal maximum level in every year, and that the tailings surfaces would always have remained flooded.

6.4 GENERAL CONCLUSIONS

The two tailings facilities are different in concept and size. As was shown in the analyses and simulations this translates directly into different levels of confidence in the hydraulic performance to be expected at each site.

The Panel mine tailings facility is a two cell arrangement, made up of two separate natural lakes. There are no internal dykes which make the design for satisfactory water balance more complex.

By comparison the Quirke mine facility has five cells in cascade, each separated by an internal dyke, and the total surface area, in relation to the drainage area, is somewhat larger, thus increasing the importance of evaporation.

The general conclusions on the adequacy of the proposed schemes at closeout are summarized as follows.

6.4.1 Quirke Basin

Water Level Control Concept

The information presented by Rio Algom on the operation of the tailings facility after closeout refers to the objective of maintaining a maximum controlled water depth of two feet over the tailings surface in each cell, except during floods. It is understood that this may be beneficial in the development of vegetation. No specific facility (weir, low level outlet etc.,) has been described to achieve this level control.

The analyses have shown that adequate freeboards are provided at intermediate and external dykes for the safety of the structures under design flood conditions. However this is predicated on the initial water levels at the start of the flood being at, or below, the maximum controlled levels.

The low flow analyses were carried out assuming that no such control procedure was in place, and thus water levels were allowed to rise naturally above the maximum controlled levels. The resulting simulations showed that the levels would be above the nominal maximum levels for most of the time.

From the results of these analyses it may also be concluded that the flooding of the tailings surfaces cannot be assured if the concept of the maximum controlled depth of 2 feet is implemented.

It is therefore considered that additional development of the flooded tailings concept is required for the Quirke tailings basin. Aside from the question of assurance in terms of the validity of estimates for seepage and evaporation, which also affect the acceptability of the Rio Algom proposal, there are two basic alternatives that have to be considered in finalizing the proposed scheme.

If the concept of a controlled maximum level is to be implemented, then provision for an unattended makeup water scheme has to be made. This may not be unduly complex, since the requirement is to add water over an extended period. However this does mean that more natural runoff will be exposed to contaminants in the

tailings basin than would otherwise be the case, and this could have downstream implications.

The alternative is to abandon this concept, in which case the basin could be expected to perform satisfactorily under very extreme low flow conditions. However with the natural water levels in the cells being raised, either the intermediate dyke weirs, and possibly the G2 dam spillway, would have to be enlarged, or dyke crests would have to be raised. It is presumed that the first alternative would be selected as little extra cost would be involved.

Seepage

A critical element in the water balance in the 5 cell basin is the amount of flow through the tailings deposits, under the intermediate dykes.

The analyses showed that if the assumed seepage rate was increased by less than 50% over the most probable value, then the upstream cells would lose their water cover. By comparison the possible range of seepage rates indicates a possible maximum rate of 1 000% over the average rate.

The evaluation of the feasibility of the proposed scheme is therefore dependant on the validity of the estimated seepage rates used in the analyses.

Rio Algom are proposing to carry out flooding tests in Cell 14, so that seepage rates may be estimated from actual measurements applied to the water balance for the cell. This is considered to be a preferred method to obtain a better estimate of the seepage rates.

The monitoring program and length of the test program will be important. Considering that seepage is only one of a number of parameters in the water balance equation, it will be necessary to develop accurate estimates of evaporation and precipitation over the period of the test. Equally the measurement of water levels should be as accurate as possible, as the surface area of the cell is large. It is expected that the test should start immediately after the spring thaw and continue over a period of 5-6 months.

Evaporation

The evaporation estimates provided by Rio Algom appear to be somewhat higher than those used in this review. Evaporation is a critical element in the assessment of the proposed scheme, and any further appraisal of this scheme should examine this aspect carefully, including possible impacts from vegetation in the cell flooded areas.

Ranges of Applicability and Assumptions

The primary approach to evaluating the proposed closeout scheme for the Quirke basin has been to simulate flows and levels using most probable scenarios, both for flood safety and low flow water supply.

With regard to the flood studies, the analyses have shown that starting levels will be important, however limiting initial levels have not been determined. Equally the volume of the flood is critical, however the analyses have been based on an estimated Probable Maximum Flood, which includes a number of assumptions to maximize the size of the flood. There is some range in possible flood

volumes, that would generally meet criteria for the PMF, as illustrated by the higher volume estimated by Rio Algom, however these have not been tested.

For the low flow analyses the studies have shown that a modest increase in the estimated seepage rate in the tailings deposit would be of critical importance, however improved estimates will depend on additional field tests, which have yet to be carried out.

The key issue appears to be the question of control of maximum depths. When this is resolved it would be advantageous to carry out a series of sensitivity analyses to determine the conditions under which the facility would operate satisfactorily.

6.4.2 Panel Basin

Technical Feasibility

1. There should be no risk of damage to internal or external dykes due to overtopping during very extreme flood events. Satisfactory freeboard margins remain under all conditions up to the maximum probable flood.

2. Under expected evaporation conditions the natural runoff will be adequate to offset evaporation and seepage losses, and thus maintain the tailings in a flooded state. This applies to average and extreme low flow conditions. This result is not sensitive to estimate of seepage at the external dykes.

Range of Applicability and Assumptions

The flood analyses did assume initial levels at the proposed controlled maximum depths, as for the Quirke basin. However the remaining freeboards are sufficiently large that a significant increase in starting levels, as could result from abandoning the maximum controlled level concept, could be accepted.

Resident storage, or water depth over the tailings surface, at the end of a very extreme dry period is sufficient that any probable increase in evaporation or external dyke seepage estimates could not result in exposure of the tailings surfaces in either cell.

ACKNOWLEDGEMENTS

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The project was carried out by staff of Shawinigan Lavalin Inc. and Geocon Inc., which are divisions of Lavalin Inc.

ANNEX A
PROJECT DOCUMENTATION

A. REPORTS PROVIDED BY RIO ALGOM/AECB

Golder Associates Reports

Golder Associates, "Geotechnical Investigation and Design Recommendations, Dam J. Quirke Tailings Basin Expansion", Report 801-1089-2, January 1981a.

Golder Associates, "Geotechnical Investigation and Design Recommendations, Dam I, Quirke Tailings Basin Expansion", Report 801-1089-4, February 1981b.

Golder Associates, "Stability of the East Dyke, Quirke Mine Tailings Management Area", Report 801-1099-12, June 1981c.

Golder Associates, "Geotechnical Investigation and Design Recommendations, Dam K_1, Quirke Tailings Basin Expansion", Report 801-1089-7, September 1981d.

Golder Associates, "Geotechnical Investigation and Design Recommendations, Berm M, Quirke Tailings Basin Expansion", Report 811-1090-1, December 1981e.

Golder Associates, "Phase 2 Status Report on the evaluation of Decommissioning Alternatives, Quirke and Panel Mine Tailings Impoundments", Report 871-1112, March 1989a.

Golder Associates, "Final Dam Design and Tailings Deposition Plan, Panel Mine Tailings Impoundment", Report 881-1917A-1, April 1989b.

Golder Associates, "Preliminary Design Brief, Main Dam Remediation and Raising, Quirke Mine Tailings Impoundment", Report 881-1819B-1, April 1989c.

Golder Associates, General Construction Specifications S400-02-10, 11, 13 and 14, Main Dam Site Preparation Drawings 591-Q720-100, 101, 102, 103 and 104.

Golder Associates Ltd. "Geotechnical Investigations and Design Recommendations - Main Dam Remediation - Quirke Mine Tailings Impoundment", March 1990.

Golder Associates Ltd. "As Built Report for Dams K_2, G_2 and J (Stage 2), Quirke Tailings Basin Expansion, December 1983.

Golder Associates Ltd. Report, No. 891-1821, August, 1989 - "Design Criteria and Construction Drawings - Proposed Raisings of Dams L and M - Quirke Mine Tailings Impoundment".

Golder Associates/MacLaren Report, - "Quirke Mine Tailings Impoundment - Volume IV - Hydrogeological Assessment of the Tailings Impoundment Basin".

Golder Associates Report No. 821-1163, October, **1992***, and Appendices - "Quirke Mine Tailings Impoundment - Hydrogeological Re-Assessment of West End of Tailings Basin".

Golder Associates Ltd. Report No. 871-1112, dated March, 1989 - "Phase 2 Status Report On The Evaluation Of Decommissioning Alternatives - Quirke and Panel Mine Tailings Impoundments".

Golder Associates, Report to Rio Algom Limited, "Geotechnical Investigation and Design Recommendations, Dam G_2, Quirke Tailings Basin Expansion Contract MP-0001, Project 262, Elliot Lake, Ontario". Report No. 11578-N, September 1981.

Golder Associates, Report to Rio Algom Limited, "Geotechnical Investigation and Design Recommendations, Dam L, Quirke Tailings Basin Expansion Contract MP-0001, Project 262, Elliot Lake, Ontario". Report No. 11578-Q, October 1981.

Golder Associates, Report to Rio Algom Limited, "Geotechnical Investigation and Design Recommendations, Berm M, Quirke Tailings Basin Expansion Contract MP-0001, Project 262, Elliot Lake, Ontario". Report No. 11578-R, December 1981.

Cumming Cockburn Reports

Brief excerpts of Quirke Mine Tailings reports.

Brief excerpts of Panel Mine Tailings reports.

MacLaren Reports

James F. MacLaren - Golder Associates, "Panel Mine Tailings Impoundment - Volume III; Design Brief, Construction of 1978 Facilities", February 1978.

Excerpts from James F. MacLaren - Golder Associates, "Panel Mine Tailings Impoundment, Volume VI: (1) Tailings Deposition Scheme, (2) Details of Tailings Effluent Treatment System", May 1979.

James F. MacLaren, "Environmental Assessment of the Proposed Elliot Lake Uranium Mines Expansion", Volume 1, Background Information, March 1977*.

James F. MacLaren, "Environmental Assessment of the Proposed Elliot Lake Uranium Mines Expansion", Volume 2, Background Information Update, February 1978*.

James F. MacLaren, "Environmental Assessment of the Proposed Elliot Lake Uranium Mines Expansion", Addendum 1, September 1978*.

MacLaren Plansearch, "Spanish River Modelling Study Low Flow Augmentation". Report to Northeastern Region Ministry of the Environment. Report No. 35225, February 1984*.

MacLaren Engineers Planners & Scientists Inc. and Golder Associates, "Quirke Mine Tailings Impoundment", Draft Report to Rio Algom Limited, Volume III, Design Brief. Report No. 11578-BB, November 1980*.

James F. MacLaren Limited and Golder Associates, Report to Rio Algom Limited On "Panel Mine Tailings Impoundment". Volume VI (1) Tailings Deposition Scheme; (2) Details of Tailings Effluent Treatment System. Report No. 10957-Q, May 1979*.

James F. MacLaren Limited and H.Q. Golder Associates, Report to Rio Algom Limited on "Quirke Mine Tailings Impoundment". Volume II, Site Selection Report. Report No. 11578-A, February 1980*.

James F. MacLaren Limited and Golder Associates, Report to Rio Algom Limited On "Quirke Mine Tailings Impoundment Stage 1 Development Works, Project Description and OPB Cost Estimate". Report No. 11578-B, July 1980*.

* Internal Lavalin reports

B. REPORTS OBTAINED BY SHAWINIGAN

M.M. Dillon, Report to Ontario Hydro "Probable Maximum Flood Mississagi River", Report No. 88150, January 1988.

M.M. Dillon, Report to Ontario Hydro "Mississagi Basin Probable Maximum Precipitation Study", September 1987.

T.T. Nguyen, Lakehead University, Thesis "An Assessment of Methods of Estimating Evaporation in the Northwest Ontario Area Using Climatological Data, August 1987.

Environment Canada Compilation of Ontario Storm Analyses by Canadian Climate Centre.

C. ENGINEERING DRAWINGS

The following engineering drawings were provided by AECB:

1. Drawing Index and Specification Index.

2. Drawing Number 591-Q720-117 - Quirke Mine Tailings Basin Construction - Main Dam, Embankment Construction, Cross-Sections.

3. Drawing Number 591-Q720-116 - Quirke Mine Tailings Basin Construction - Main Dam, Embankment Construction, Typical Cross-Section and Cross-Sections.

4. Drawing Number 591-Q720-113 - Quirke Mine Tailings Basin Construction - Main Dam, Embankment Construction, General Arrangement Plan (1989 Construction by Others).

5. Drawing Number 591-Q720-114 - Quirke Mine Tailings Basin Construction - Main Dam, Embankment Construction, Detailed General Arrangement Plan (1990 Construction).

6. Drawing Number 591-Q720-115 - Quirke Mine Tailings Basin Construction - Main Dam, Embankment Construction, Centreline Profile, Typical Cross-Section & Details.

7. Drawing Number 591-Q720-118 - Quirke Mine Tailings Basin Construction - Main Dam, Embankment Construction, Till Borrow Area, Development Plan and Cross-Sections.

8. Drawing Number 262-720-177 - Quirke Tailing Basin Expansion - Dams, Channels & Decant Dam G_2, General Arrangement Plan and Details.

9. Drawing Number 262-720-178 - Quirke Tailing Basin Expansion - Dams, Channels & Decant Dam G_2, Foundation Preparation Plan.

10. Drawing Number 262-720-179 - Quirke Tailing Basin Expansion - Dams, Channels & Decant Dam G_2, Cross Sections.

11. Drawing Number 262-720-195 - Quirke Tailing Basin Expansion - Dams, Channels & Decant Dam J - Stage 2 Construction, General Arrangement.

12. Drawing Number 262-720-197 - Quirke Tailing Basin Expansion - Dams, Channels & Decant Dam J - Stage 2 Construction, Cross Sections.

13. Drawing Number 262-720-198 - Quirke Tailing Basin Expansion - Dams, Channels & Decant Dam J - Stage 2 Construction, Cross Sections

14. Drawing Number 262-720-185 - Quirke Tailing Basin Expansion - Dams, Channels & Decant Dam "K_2", Cross Sections - Sheet 1.

15. Drawing Number 262-720-186 - Quirke Tailing Basin Expansion - Dams, Channels & Decant Dam "K_2", Cross Sections - Sheet 2.

16. Drawing Number 262-720-184 - Quirke Tailing Basin Expansion - Dams, Channels & Decant Dam "K_2", Foundation Preparation Plan.

17. Drawing Number 262-720-183 - Quirke Tailing Basin Expansion - Dams, Channels & Decant Dam "K_2", General Arrangement.

18. Rio Algom Limited - Quirke Waste Management Area - May 89 Mapping.

19. Rio Algom Panel Mine, May 1989.

D. FLOW RECORDS

Available daily flows for the following stations were obtained from Water Survey of Canada.

Station	Drainage Area Km^2	No.	Record Start Date
Serpent River at Dunlop Lake	109	02CD002	77/02
Serpent River above Quirke Lake	157	02CD006	67/10
Serpent River below Quirke Lake	319	02CD002	77/10
Rochester Creek above Quirke Lake	99.5	02CD005	77/09
Little Nordic Creek at Elliot Lake	4.58	02CD007	82/12
Nordic Mine Tailings Ditch (Site 1)	2.09	02CD008	*84/09
Nordic Mine Tailings Ditch (Site 2)	2.76	02CD009	*84/04
Junction Creek below Kelley Lake	207	02CF012	77/01
Moose Creek at Levack	40.8	02CF013	81/03
Vermilion River near Val Caron	704	02CF011	70/06
Whitson River at Chelmsford	243	02CF007	60/08
Whitson River at Val Caron	155	02CF008	60/11

* Intermittent or incomplete

The location of these gauges is shown in the main text - Figure 4.1.

E. MAPPING

Energy, Mines and Resources

Maps of the national Topographic System of Canada

Index Number: 1

Map	Number	Title	Scale	Date
41	I	Sudbury, Ontario	1:250 000	1960, Edition 1
41	I	Sudbury, Ontario	1:250 000	1977, Edition 2
41	I/4	Whitefish Falls, Ontario	1:50 000	1975, Edition 3
41	I/5	Espanola, Ontario	1:50 000	1975, Edition 3
41	I/10 (2 copies)	Capreol, Ontario	1:50 000	1988, Edition 5
41	I/10E	Capreol, Ontario	1:50 000	1960, Edition 2
41	I/11 (2 copies)	Chelmsford, Ontario	1:50 000	1988, Edition 5
41	I/11E	Chelmsford, Ontario	1:50 000	1963, Edition 2
41	I/11W	Chelmsford, Ontario	1:50 000	1963, Edition 2
41	I/14 (2 copies)	Venetian Lake, Ontario	1:50 000	1975, Edition 2
41	I/15 (2 copies)	Milnet, Ontario	1:50 000	1975, Edition 3
41	J	Blind River, Can-U.S.	1:250 000	1985, Edition 3
41	J/1	Spanish, Ontario	1:50 000	1975, Edition 3
41	J/7	Elliot Lake, Algoma District, Ontario	1:50 000	1975, Edition 2
41	J/9	Madawanson Lake, Ontario	1:50 000	1977, Edition 2
41	J/10	Rawhide Lake, Algoma District, Ontario	1:50 000	1977, Edition 2
41	K	Sault Ste. Marie, Can-U.S.	1:250 000	1963, Edition 1
41	K	Sault Ste. Marie, Can-U.S.	1:250 000	1989, Edition 3
41	P/2 (2 copies)	Pilgrim Creek, Sudbury District, Ontario	1:50 000	1976, Edition 2
41	P/3 (2 copies)	Thor Lake, Sudbury District, Ontario	1:50 000	1976, Edition 2

Ontario Ministry of Natural Resources

ZONE 17 SCALE: 1:10 000 DATE: 1979 INDEX MAP NUMBER: 11

SHEETS:

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ZONE 17 SCALE: 1:10 000 DATE: 1979 INDEX MAP NUMBER: 11

SHEETS:				DATE:
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20	17	3800	51400	1979
20	17	3800	51500	1983